STRAIN BASED PERFORMANCE EVALUATION CHARTS FOR RECTANGULAR REINFORCED CONCRETE COLUMNS IN NONLINEAR ANALYSIS OF STRUCTURES

by

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ABSTRACT

Several methods and criterion are being used recently in determination of the deformations occurred in structural elements of a building and performance evaluation of these structural elements under earthquake motion using nonlinear analysis.

Performance evaluation of structures under nonlinear earthquake loading using material strains, rather than the rotations as commonly used in other seismic codes (*FEMA356, ATC40, EUROCODE8* etc.) is the basic subject of this study. Performance evaluation using material strains is also being mentioned in *Chapter 13 of New Turkish Seismic Code Draft*.

For performance evaluation using member strains, *Strain Based Performance Evaluation Charts* for rectangular reinforced concrete columns have been formed which bring easiness by omitting several cross sectional analysis in order to transform rotations into strains.

These *Strain Based Performance Evaluation Charts* are intended to be very useful for the structural engineers who will practice nonlinear analysis using *New Turkish Seismic Code* in the future. However it should be noticed that these charts are formed only for rectangular reinforced columns with limited concrete-steel grade combinations. Therefore further studies should be performed for other combinations and beams.

ÖZET

Son yıllarda geliştirilen ve hala gelişmekte olan, deprem etkisi alında mevcut yapıların elastik olmayan (lineer olmayan – nonlineer) analizinde, yapı elemanlarında meydana gelen deformasyonların belirlenmesi ve bu elemanların deprem etkisi altındaki performanslarının belirlenmesinde çeşitli yöntemler ve kriterler kullanılmaktadır.

Bu çalışmada, diğer ülke yönetmeliklerinden (*FEMA356, ATC40, EUROCODE8*) farklı olarak *Yeni Türk Deprem Yönetmeliği Taslak Çalışması Bölüm 13*'te de yer alan, plastik mafsal dönmeleri yerine malzeme birim şekil değiştirmelerine göre performans değerlendirmesi aşamasında *Yeni Türk Deprem Yönemeliğini* kullanacak uygulamacı mühendislere büyük katkı sağlayacağı düşünülen, taşıyıcı sistem elemanları için kesit birim şekil değiştermesine dayalı performans değerlendirme grafikleri oluşturulmuştur.

Elde edilen bu grafikler, uygulayıcı mühendisin eleman performansını belirlenmesinde dönme değerlerinden şekil değiştirmeye geçerken farklı eksenel kuvvet seviyeleri için farklı kesit analizi yapmaksızın sonuç almasında yararlı olacaktır. Ama unutulmamalıdır ki bu grafikler sadece dikdörtgen betonarme kolonlar ve kısıtlı beton- çelik dayanımı kombinasyonları için hazırlanmıştır. Kirişler ve farklı kombinasyonlar için bu çalışma geliştirlmelidir.

TABLE OF CONTENTS

ACKNOWLEDGEMENTS .... *iii*

Abstract .... *iv*

Özet .... *v*

LIST OF FIGURES .... *vii*

LIST OF SYMBOLS/ABBREVIATIONS .... *xi*

1. INTRODUCTION *1*

1.1. General **1**

1.2. The Basics of Nonlinear Analysis in New Turkish Seismic Code Draft **2**

2. STRAIN BASED PERFORMANCE EVALUATION CHARTS FOR RECTANGULAR REINFORCED CONCRETE COLUMNS *7*

2.1. Introduction **7**

2.2. Analysis Procedure **13**

2.3. Effects of Different Parameters on the Performance Evaluation Charts **17**

2.4. Performance Evaluation Charts For Different Concrete & Steel Grade Combinations **35**

2.5. Comparison of Strain Based Performance Evaluation Charts with *FEMA356* Performance Evaluation Criterion **68**

3. A practical Approach in Calculation of Yield Rotations *75*

4. EXAMPLES *76*

4.1. Performance Evaluation of An Old Existing Building by Using Strain Based Performance Evaluation Charts **76**

4.2. Performance Evaluation of a Recently Well Designed Building Using Strain Based Performance Evaluation Charts **103**

5. CONCLUSION *130*

REFERENCES *133*

LIST OF FIGURES

Figure 1.1. Acquiring modal displacement demand for $T_1 \geq T_B$ 4

Figure 1.2. Acquiring modal displacement demand for $T_1 < T_B$ 6

Figure 2.1. Mander confined and unconfined concrete model 8

Figure 2.2. Park reinforcing steel model 9

Figure 2.3. Typical Analysis Cross Section 10

Figure 2.4. Limiting strains for unconfined sections 11

Figure 2.5. Limiting strains for confined sections. 12

Figure 2.5. Limiting strains for semi confined sections. 12

Figure 2.7. Effects of section dimensions for unconfined sections 18

Figure 2.8. Effects of section dimensions for confined sections 20

Figure 2.9. Effects of confinement levels 24

Figure 2.10. Effects of reinforcement ratios for unconfined sections 26

Figure 2.11. Effects of reinforcement ratios for confined sections 28

Figure 2.12. Effects of concrete grades for unconfined sections 30

Figure 2.13. Effects of concrete grades for confined sections 32

Figure 2.14. Effects of reinforcing steel grades for unconfined sections............................34

Figure 2.15. Several performance evaluation charts for unconfined sections for different concrete-steel grade combinations 53

Figure 2.16. Several performance evaluation charts for semi confined sections for different concrete-steel grade combinations..60

Figure 2.17. Several performance evaluation charts for full confined sections for different concrete-steel grade combinations 67

Figure 2.18. Comparison of strain based performance evaluation charts with FEMA356 performance evaluation criterion for different performance levels for unconfined (non conforming) sections for S220 grade reinforcement steel. ...70

Figure 2.19. Comparison of strain based performance evaluation charts with FEMA356 performance evaluation criterion for different performance levels for unconfined (non conforming) sections for S420 grade reinforcement steel. ...72

Figure 2.20. Comparison of strain based performance evaluation charts with FEMA356 performance evaluation criterion for different performance levels for confined (conforming) sections for S420 grade reinforcement steel. 74

Figure 4.1. Typical floor plan 77

Figure 4.2. Elastic Response Spectrum (TSC98)..78

Figure 4.3. 3D model of the structure 79

Figure 4.4. X directional pushover curve...81

Figure 4.5. X directional capacity diagram..82

Figure 4.6. Acquiring modal displacement demand 83

Figure 4.7. Acquiring performance displacement for X direction 83

Figure 4.8. Hinge locations for X directional pushover analysis..84

Figure 4.9. Performance evaluation charts for X directional pushover analysis 91

Figure 4.10. Y directional pushover curve 92

Figure 4.11. Y directional capacity diagram..93

Figure 4.12. Acquiring performance displacement for Y direction 94

Figure 4.13. Hinge locations for Y directional pushover analysis......................................95

Figure 4.14. Performance evaluation charts for Y directional pushover analysis 101

Figure 4.15. Typical floor plan 103

Figure 4.16. Elastic response spectrum (TSC98)...105

Figure 4.17. 3D model of the structure 106

Figure 4.18. X directional pushover curve..107

Figure 4.19. X directional capacity diagram...108

Figure 4.20. Acquiring performance displacement for X direction 109

Figure 4.21. Hinge locations for X directional pushover analysis...................................110

Figure 4.22. Performance evaluation charts for X directional pushover analysis 117

Figure 4.23. Y directional pushover curve 118

Figure 4.24. Y directional capacity diagram 119

Figure 4.25. Acquiring performance displacement for Y direction 120

Figure 4.26. Hinge locations for Y directional pushover analysis 121

Figure 4.27. Performance evaluation charts for Y directional pushover analysis 128

LIST OF SYMBOLS/ABBREVIATIONS

A_g	= Column section area
A_{sh}	= Reinforcement steel area
$a_1^{(i)}$	= 1st mode modal pseudo acceleration acquired at the end of (i)th pushover step
$a_{y,1}$	= 1st mode equivalent yield pseudo
$C_{R,1}$	= 1st mode displacement
CP	= Collapse prevention performance level
d	= Effective section depth
d	= Cover concrete depth
$d_1^{(i)}$	= 1st mode modal displacement acquired at the end of the (i)th step of pushover analysis
E_c	= Concrete elasticity modulus
EI_o	= Bending rigidity of the uncracked section
f_l	= Confining stress
f_{ck}	= Characteristic concrete compression strength
f_{yh}	= Transverse reinforcement yield stress
f_{yk}	= Characteristic reinforcement steel strength
f_{ywk}	= Transverse reinforcement yield stress
IO	= Immediate occupation performance level
K_e	= Confinement effectiveness ratio
L_p	= Plastic hinge length
LS	= Life Safety performance level
N_d	= Vertical load originated axial force
s	= Transverse reinforcement spacing along the height of the column
$S_{ae,1}$	= 1st mode elastic spectral acceleration
$S_{de,1}$	= 1st mode elastic spectral displacement

$S_{di,1}$	= 1st mode inelastic (nonlinear) spectral displacement
T_B	= Characteristic period in acceleration spectrum defined in Turkish Seismic Code 1998 Section 6.4.3
T_1	= 1st natural vibration period
ε_{cc}	= Concrete compression strain on the outer bound of the confined core concrete
ε_c	= Concrete compression strain on the outermost bound fiber of the section
ε_s	= Strain in the reinforcement steel
Φ_y	= Yield curvature
Φ_p	= Plastic curvature
Φ_t	= Total curvature
Γ_{x1}	= 1st mode contribution multiplier in x directional earthquake motion
$\omega_1^{(i)}$	= 1st mode (dominant mode) natural circular frequency at the first step (i=1)
ρ_s	= Volumetric ratio of the transverse reinforcement with 135^0 hooks.
ATC	= Applied Technology Council
FEMA	= Federal Emergency Management Agency
SAP2000	= Structural Analysis Program
TSC'98	= Turkish Seismic Code 1998
XTRACT	= Cross Section Analysis Program for Structural Engineers

1. INTRODUCTION

1.1. General

Several methods and criterion are being used recently in determination of the deformations occurred in structural elements of a building and performance evaluation of these structural elements under earthquake motion using nonlinear analysis. The structural project engineers face several problems in evaluation of the plastic deformations occurred in frame elements of the structure according to the plastic hinge hypothesis.

Performance evaluation using material strains, rather than the rotations as commonly used in other seismic codes *(FEMA356, ATC40, EUROCODE8 etc)* is the basic subject of this study. Performance evaluation using material strains is also mentioned in *Chapter 13 of New Turkish Seismic Code Draft.*

For performance evaluation using member strains, *Strain Based Performance Evaluation Charts* for rectangular reinforced concrete columns have been formed which bring easiness by omitting several cross sectional analysis to transform rotations into strains.

Performance evaluation of the structural elements can be easily done by using *Strain Based Performance Evaluation Charts* by comparing the total hinge rotations formed after nonlinear analysis with the limits of the regarding performance level for acting axial force ratio.

These *Strain Based Performance Evaluation Charts* are intended to be very useful for the structural engineers who will practice nonlinear analysis using *New Turkish Seismic Code* in the future. However it should be noticed that these charts have been formed only for rectangular reinforced columns with limited concrete-steel grade combinations. Therefore further studies should be performed for other combinations and beams.

1.2. The Basics of Nonlinear Analysis in New Turkish Seismic Code Draft

The aim of the performance evaluation of the buildings under earthquake loads and nonlinear analysis methods to be used in strengthening analysis is, to obtain the plastic deformation demands in response to ductile behavior, and internal force demands in response to brittle response. Afterwards these demands will be compared with the pre-defined deformation and internal force capacities and performance evaluation will be done in base of sections and structure.

1.2.1. The Steps of Performance Evaluation using Incremental Pushover Analysis

a) The idealization of the nonlinear behavior of the structural system elements, and preparation of the analysis model.
b) Preparation of Capacity Diagram with "modal displacement - modal acceleration" as the coordinates.
c) Specification of modal displacement demand for the dominating mode using capacity diagram and modified elastic response spectrum.
d) Calculation of displacement, plastic hinge rotations and frame forces for modal displacement demand.
e) Calculation of plastic curvature demands from plastic hinge rotations and obtaining the total curvature demands and acting steel and concrete strain demands.
f) Performance evaluation will be done by comparing these strain demands with the strain capacities of steel and concrete specified for various damage levels. And also performance evaluation for brittle response will be done by comparing the shear force demand with the shear capacities.
g) All of the steps defined will be done for both earthquake directions independently.

Strain Based Performance Evaluation Charts bring easiness for the step e. By using *Strain Based Performance Evaluation Charts*, there will be no need to calculate steel and concrete strain demands for different acting axial forces. By directly using total rotations and axial force ratios, performance evaluation will be easily done for the reinforced concrete sections.

1.2.2. Idealization of Nonlinear Response

- Plastic Hinge Hypothesis will be used.
- Plastic deformation region called as plastic hinge length L_p, will be the half length of effective depth (d).

$$L_p = 0.5d \tag{1.1}$$

- Cracked stiffness of the bending elements will be calculated as follows
  a) Beams: $0.40\ EI_o$ (1.2)
  b) Columns and Shear Walls, $0.40\ EI_o$ if $N_d / (A_g f_{ck}) \leq 0.10$ (1.3)
  $0.80\ EI_o$ if $N_d / (A_g f_{ck}) \geq 0.40$ (1.4)

Axial compression force N_d, will be calculated under vertical loads and linear interpolation will be done for the values for the mid values.

1.2.3. Determination of Modal Displacement Demand in Pushover Analysis

i. Compare the dominating period T_1 of the linearly elastic system with the T_B *(TSC98 Section 6.4.4)* value.

- If $T_1 \geq T_B$ then nonlinear spectral displacement $S_{di,1}$ is equal to linear elastic displacement of the equivalent linear system $S_{de,1}$ according to the equal displacement rule.

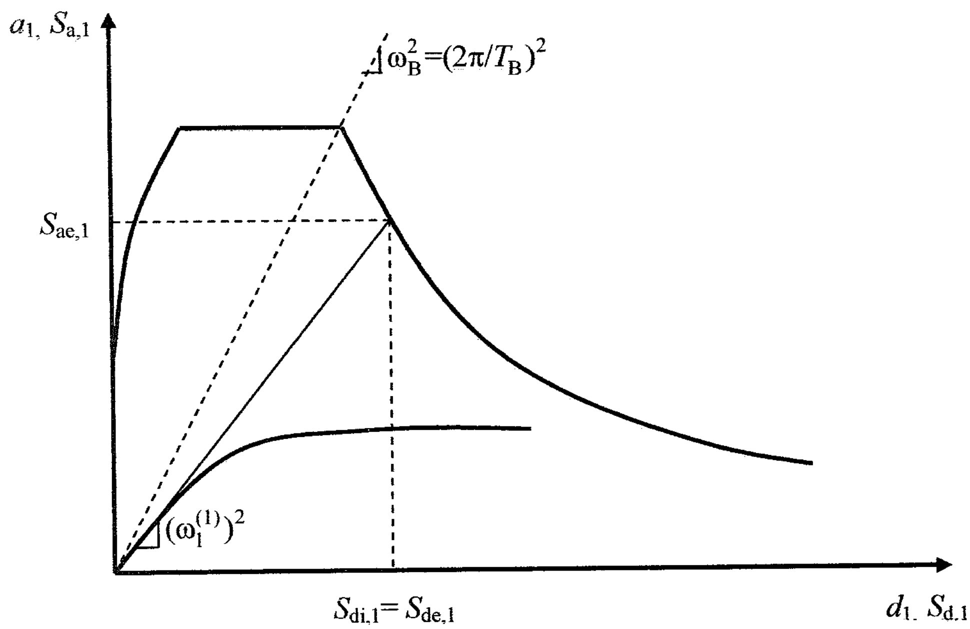


Figure 1.1. Acquiring modal displacement demand for $\boldsymbol{T_1 \geq T_B}$

- If $T_1 < T_B$ then the equation below will be executed.

$$S_{di,1} = C_{R,1}\, S_{de,1} \tag{1.5}$$

Displacement ratio $C_{R,1}$ will be calculated for every pushover step (i) with consecutive approach.

a) Modal Capacity diagram will be bilinearized as shown in *figure 1.2(a)*. The slope of the starting line will be equal to the slope of the line at first step of the pushover analysis referring the dominating mode.

b) In the first step of the consecutive approach, $C_{R,1}$ value will be chosen equal to 1, and the coordinates of equivalent yield point will be found using equal areas rule.

By taking into consideration the $a_{y,1}^{o}$ value in *figure 1.2(a)* $C_{R,1}$ is defined as below.

$$C_{R,1} = \frac{1 + (R_{y,1} - 1)\, T_B / T_1}{R_{y,1}} \geq 1 \tag{1.6}$$

$R_{y,1}$ is the strength reduction factor for the first mode.

$$R_{y,1} = \frac{S_{ae,1}}{a_{y,1}} \tag{1.7}$$

c) The coordinates of equivalent yield point will be calculated again as shown in *figure 1.2(b)*, and as a result $a_{y,1}$, $R_{y,1}$ and $C_{R,1}$ will be recalculated. If the values in consecutive two steps become close in an acceptable ratio then consecutive approach will be ended.

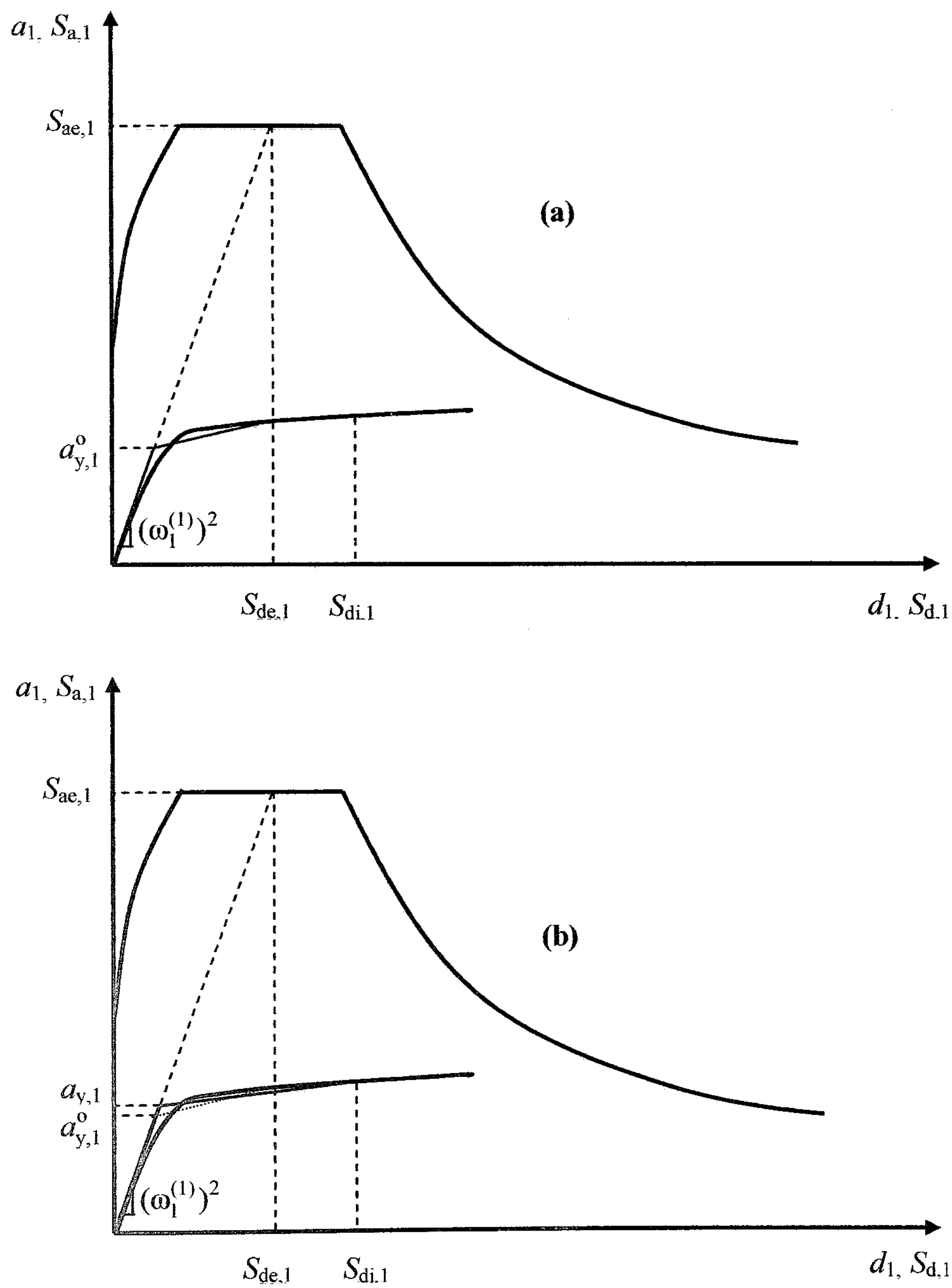


Figure 1.2. Acquiring modal displacement demand for $T_1<T_B$

2. STRAIN BASED PERFORMANCE EVALUATION CHARTS FOR RECTANGULAR REINFORCED CONCRETE COLUMNS

2.1. Introduction

Strain Based Performance Evaluation Charts have been formed to help engineers evaluate the seismic performance of existing buildings after nonlinear analysis. After performing nonlinear pushover analysis, by the help of these charts, total element rotations developed on the rectangular columns can easily be compared with the limits of the regarding performance level , omitting several cross sectional analysis to transform the rotations into material strains.

Strain Based Performance Evaluation Charts have been constructed by performing axial load – bending moment interaction analysis for each performance level and the curvature of the cross section for each axial load level is determined. The axial force - curvature charts specified for each section type have been converted to dimensionless scale by dividing the axial loads by section area and characteristic concrete strength, and multiplying the curvature values by plastic hinge length.

During cross sectional analysis to form performance evaluation charts several parameters have been taken into account such as concrete strength, steel strength, steel ratio, confinement and section dimension characteristics.

Concrete Strength

Three grades of concrete quality ($C10\ f_{ck}=10\ MPa$, $C14\ f_{ck}=14MPa$, $C20\ f_{ck}=20\ MPa$) for unconfined sections and two grades of concrete quality ($C20\ f_{ck}=20MPa$, $C25\ f_{ck}=25MPa$) for confined sections have been considered. These concrete strengths are 28 day characteristic cylinder strengths. *Mander* Confined and Unconfined Concrete Models have been used to specify concrete stress-strain relationship as shown in the chart below and described in *New Turkish Seismic Code Draft Chapter 13B*.

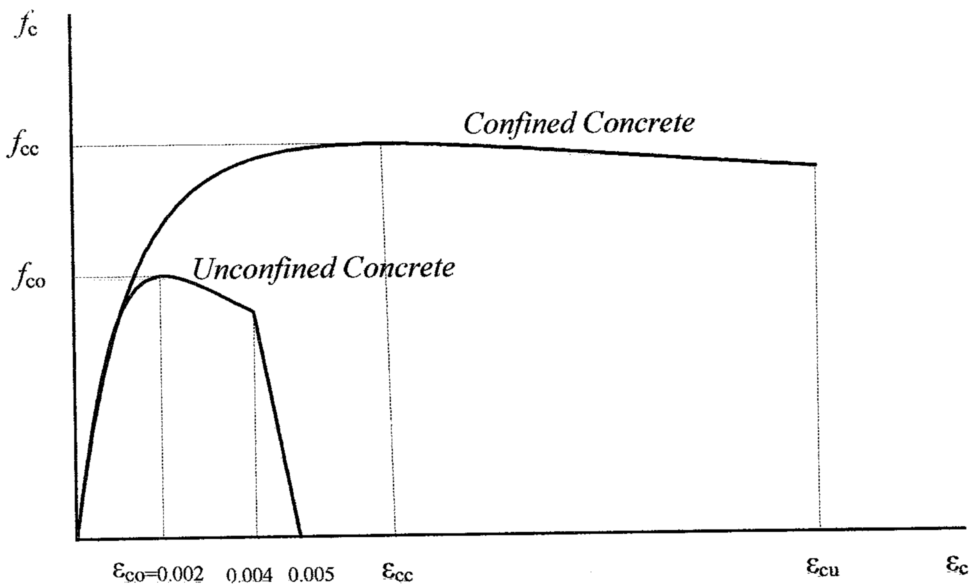


Figure 2.1. Mander confined and unconfined concrete model

Steel Strength

Two grades of steel qualities (*S420* f_{yk}*=420MPa, S220* f_{yk}*=220MPa*) have been used in unconfined section analysis. In the past years, commonly, *S220* grade steel was being used for construction purposes, however, recent years, *S420* grade steel is being used nearly at every construction. *Park* Model has been used for steels during analysis as mentioned in *New Turkish Seismic Code Draft Chapter 13B*. Stress-Strain relations for *Park* Model is shown below.

Grade	f_{yk} (Mpa)	ε_{sy}	ε_{sh}	ε_{su}	f_{su} (Mpa)
S220	220	0.0011	0.011	0.16	275
S420	420	0.0021	0.008	0.10	550

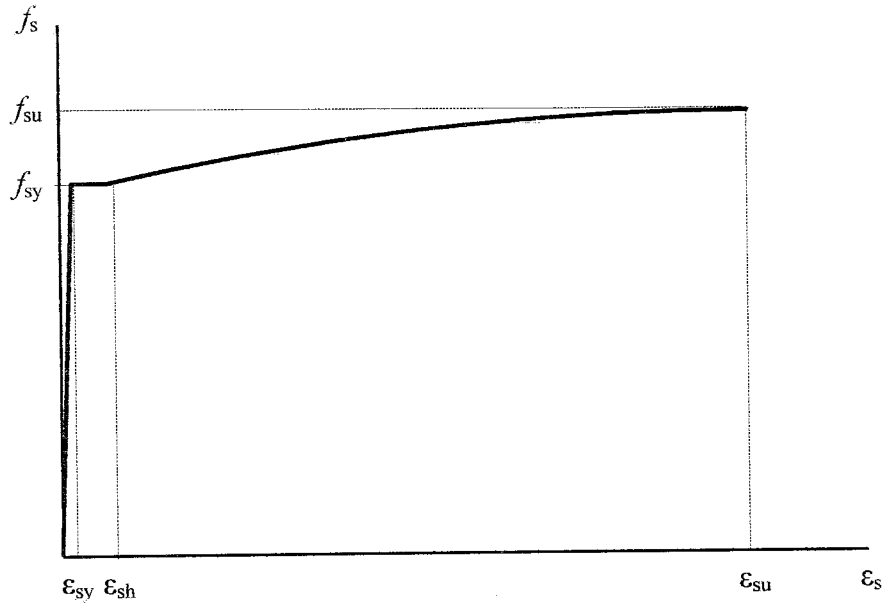


Figure 2.2. Park reinforcing steel model

Cross Section

All the sections analyzed in this study have constant reinforcement configuration with "8 bars" and the location of the bars are determined for constant *d'/d* ratio of 0.08 as shown below.

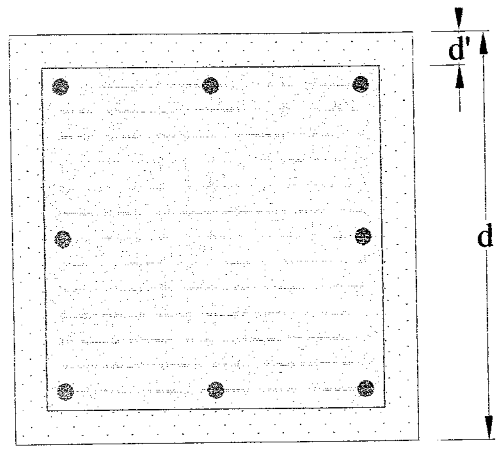


Figure 2.3. Typical Analysis Cross Section

Limiting Strains

The strain limits used in analysis for each performance level is given below for both confined and unconfined sections.

i. <u>*Unconfined Sections*</u>

	Immediate Occupancy	Life Safety	Collapse Prevention
Cover Conc.	0.004	-	-
Core Conc.	-	0.004	0.004
Steel	0.01	0.04	0.06

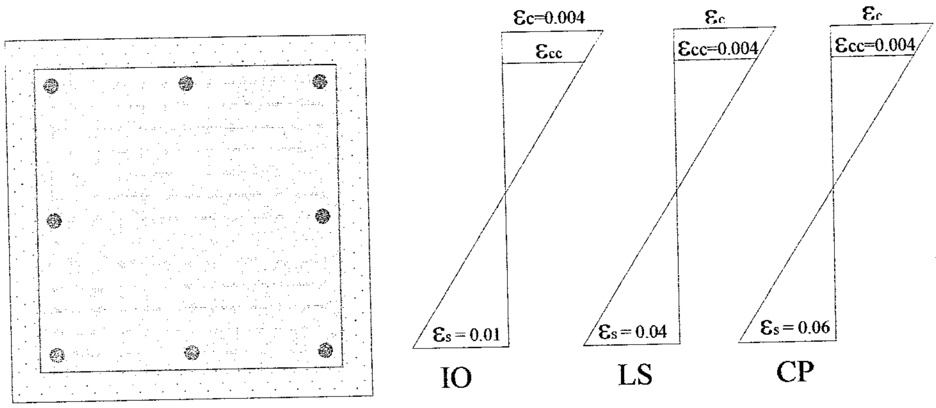


Figure 2.4. Limiting strains for unconfined sections

ii. *Confined Sections*

	Immediate Occupancy	**Life Safety**	**Collapse Prevention**
Cover Conc.	0.004	-	-
Core Conc.	-	0.0135	0.018
Steel	0.01	0.04	0.06

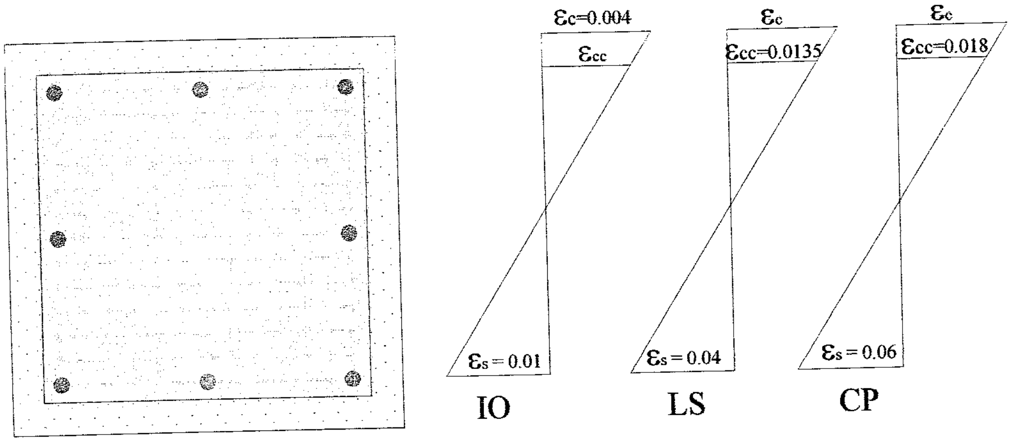


Figure 2.5. Limiting strains for confined sections.

iii. *Semi Confined Sections*

	Immediate Occupancy	Life Safety	Collapse Prevention
Cover Conc.	0.004	-	-
Core Conc.	-	0.00875	0.011
Steel	0.01	0.04	0.06

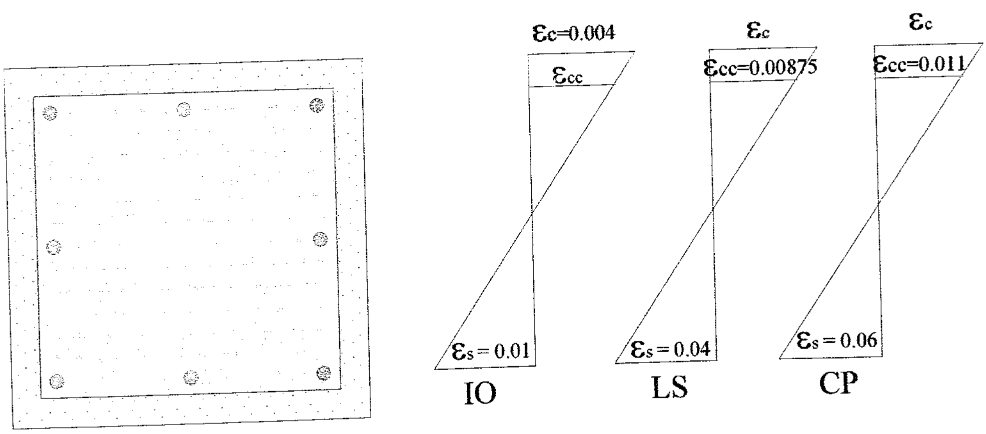


Figure 2.5. Limiting strains for semi confined sections.

2.2. Analysis Procedure

Strain Based Performance Evaluation Charts have been constructed by performing axial load – bending moment interaction analysis for each performance level and the curvature of the cross section for each axial load level has been determined. The axial force - curvature charts, specified for each section type have been converted to dimensionless scale by dividing the axial loads by section area and characteristic concrete strength, and multiplying the curvature values by plastic hinge length. Plastic hinge length has been taken as half of the effective depth of the section as in the *New Turkish Seismic Code Draft*

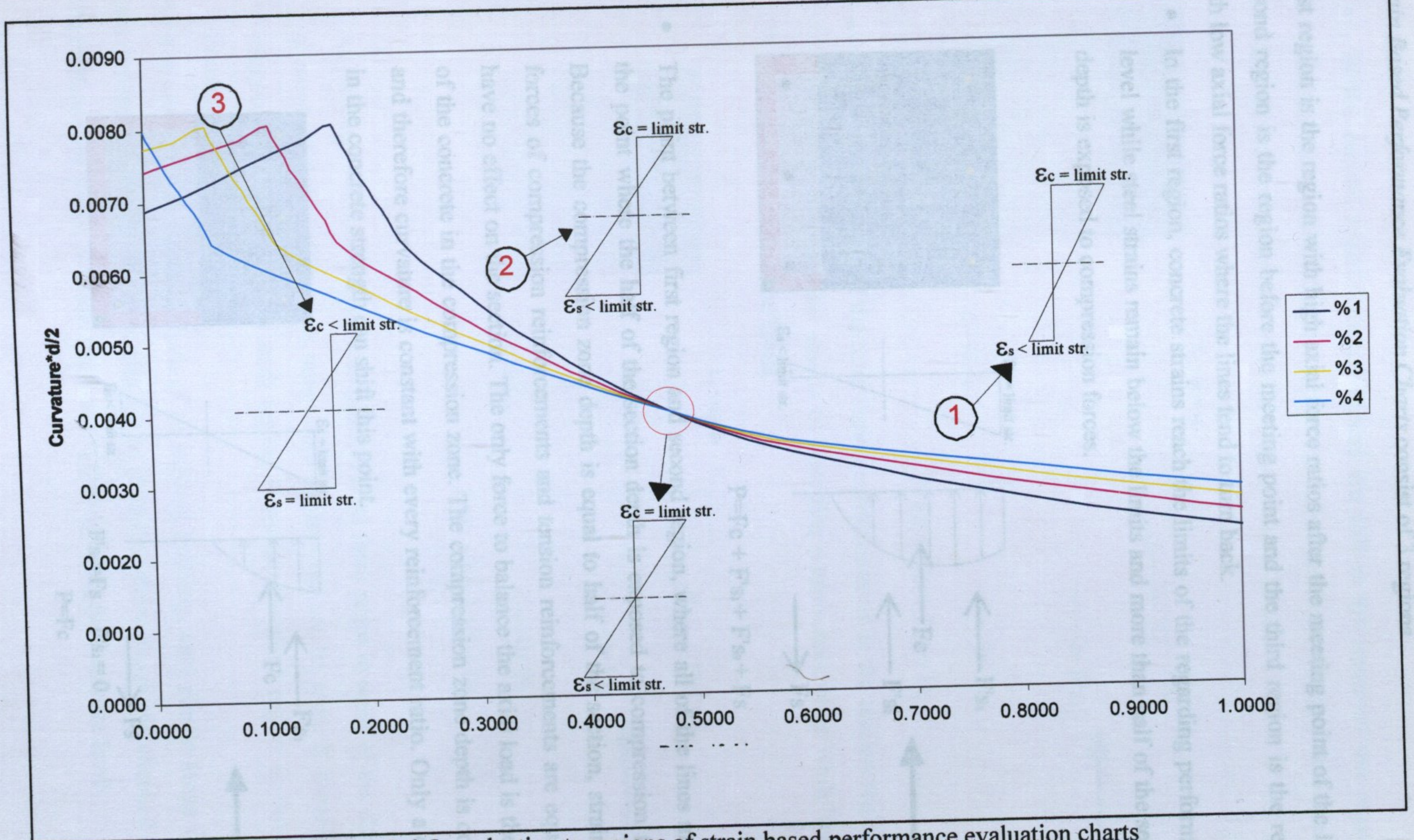
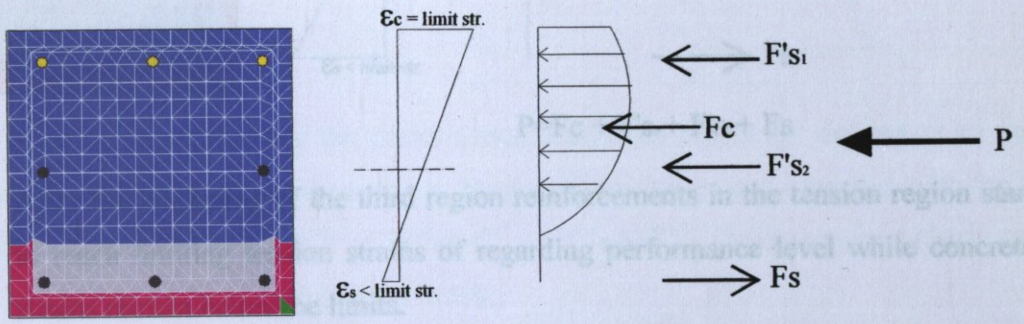


Figure 2.6. Introduction to regions of strain based performance evaluation charts

Strain Based Performance Evaluation Charts consist of 3 regions

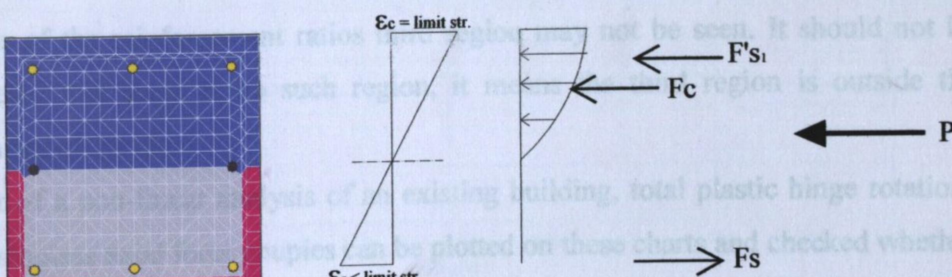
First region is the region with high axial force ratios after the meeting point of the lines, second region is the region before the meeting point and the third region is the region with low axial force ratios where the lines tend to turn back.

- **In the first region, concrete strains reach the limits of the regarding performance level while steel strains remain below the limits and more than half of the section depth is exposed to compression forces.**



$$P = Fc + F's_1 + F's_2 + Fs$$

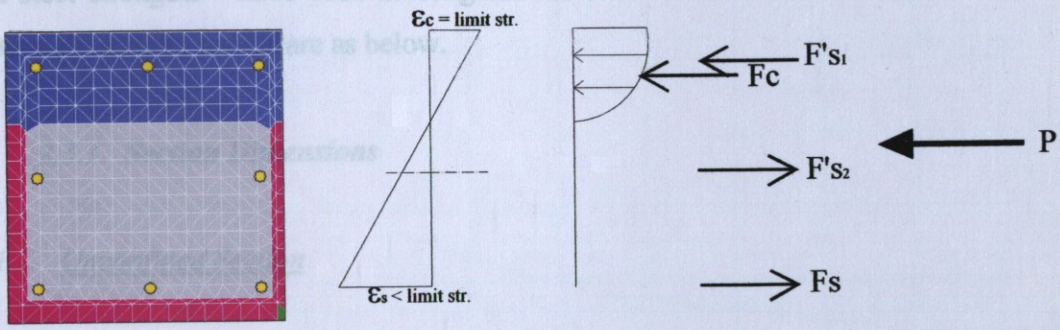
- **The point between first region and second region, where all of the lines meet, is the point where the half of the section depth is exposed to compression forces. Because the compression zone depth is equal to half of the section, strains and forces of compression reinforcements and tension reinforcements are equal and have no effect on the section. The only force to balance the axial load is the force of the concrete in the compression zone. The compression zone depth is constant and therefore curvature is constant with every reinforcement ratio. Only a change in the concrete strength can shift this point.**



$F's_1 = Fs \quad F's_2 = 0$

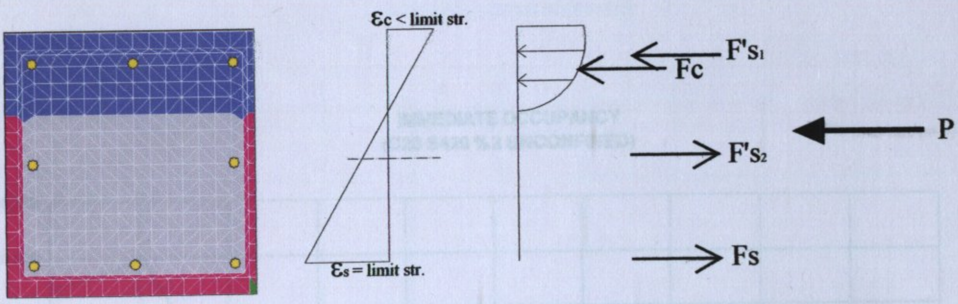
$P = Fc$

- In the second region concrete strains are still at the limits of the regarding performance level while steel strains are below the limits but this time less than half of the section depth is exposed to compression forces.



$$P = F_c + F'_{s1} + F'_{s2} + F_s$$

- After the beginning of the third region reinforcements in the tension region start to reach limiting tension strains of regarding performance level while concrete strains remain below the limits.



$$P = F_c + F'_{s1} + F'_{s2} + F_s$$

As the chart x-axis is limited with positive axial force ratios in some of the charts and at some of the reinforcement ratios third region may not be seen. It should not be interpreted that there is no such region, it means the third region is outside the boundaries of the chart.

As a result of a non-linear analysis of an existing building, total plastic hinge rotations and dimensionless axial force couples can be plotted on these charts and checked whether the rotations are above or below the limits of the regarding performance level.

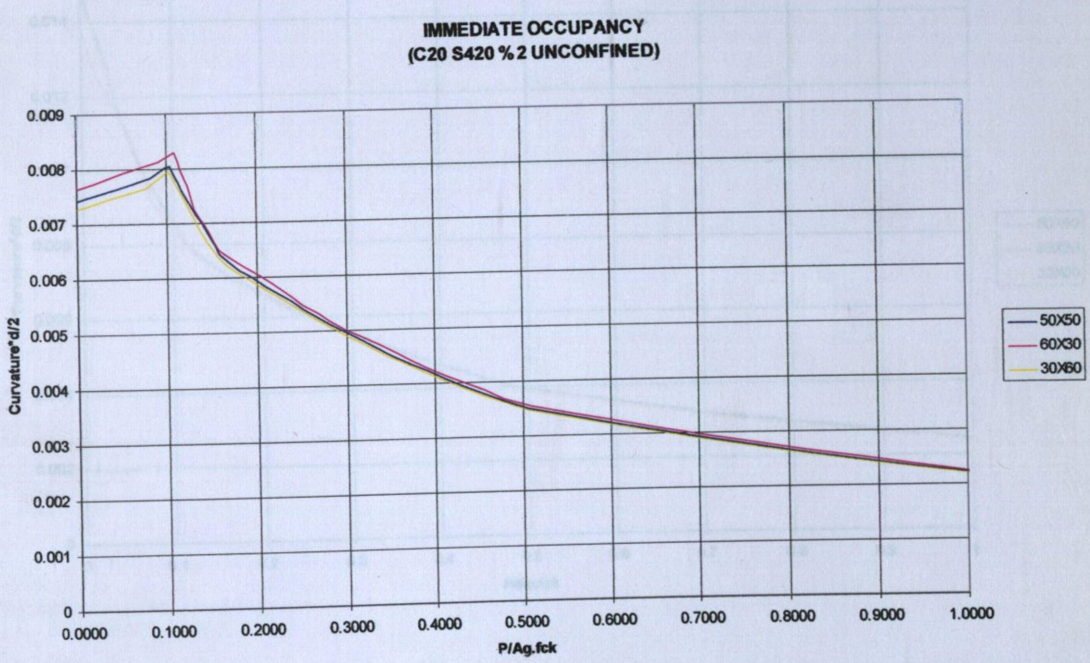
2.3. Effects of Different Parameters on the Performance Evaluation Charts

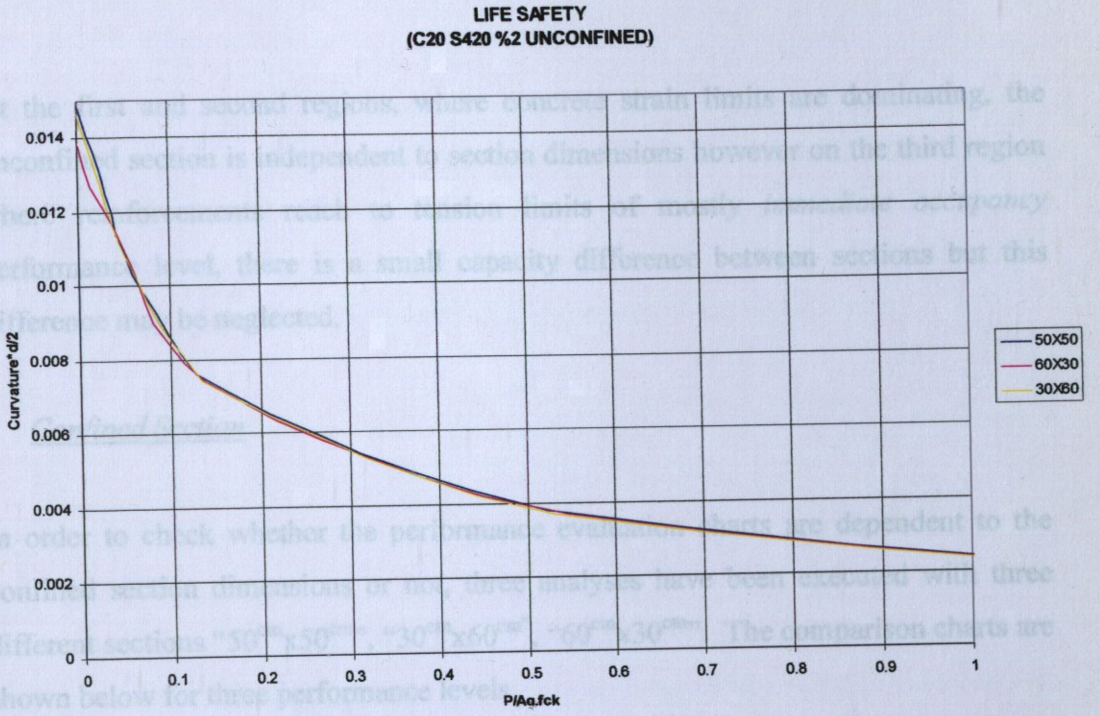
Effect of five parameters "section dimensions, confinement, reinforcement ratio, concrete and steel strengths" have been investigated for both confined and unconfined sections separately and the results are as below.

2.3.1. Section Dimensions

i. <u>*Unconfined Section*</u>

In order to check whether the performance evaluation charts are dependent to the unconfined section dimensions or not, three analyses have been executed with three different sections "50^{cm}x50^{cm}", "30^{cm}x60^{cm}", "60^{cm}x30^{cm}". The comparison charts are shown below for three performance levels.





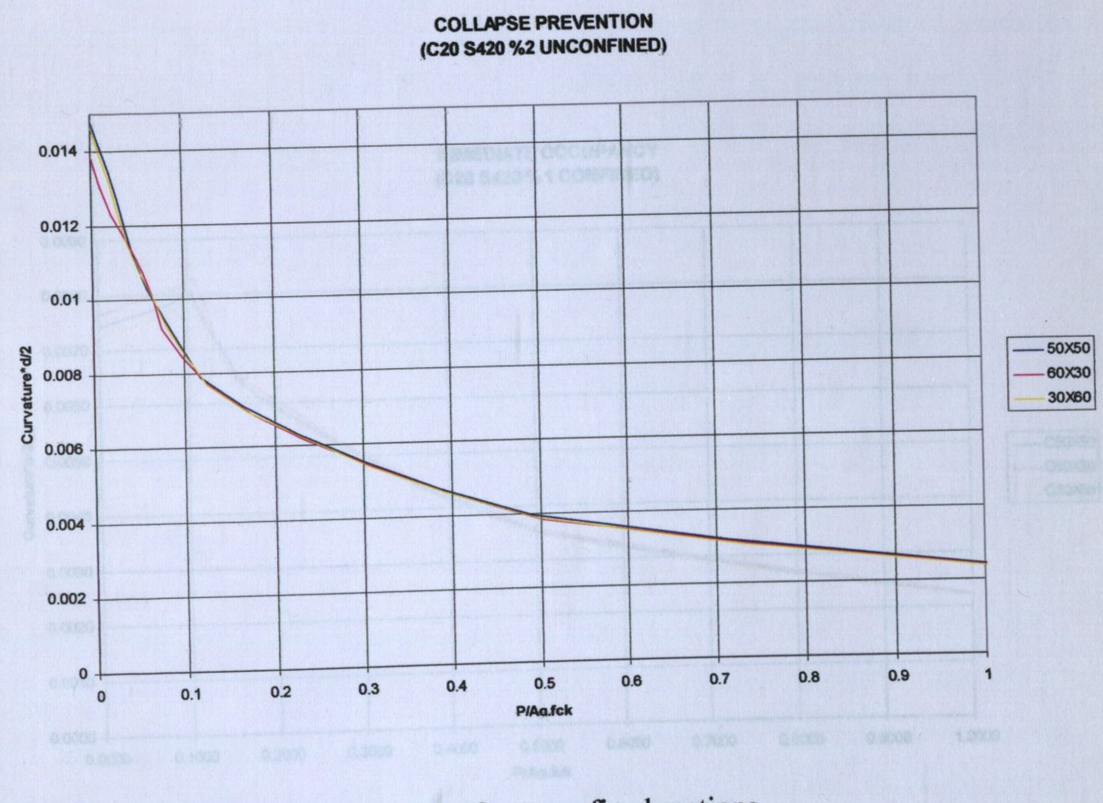
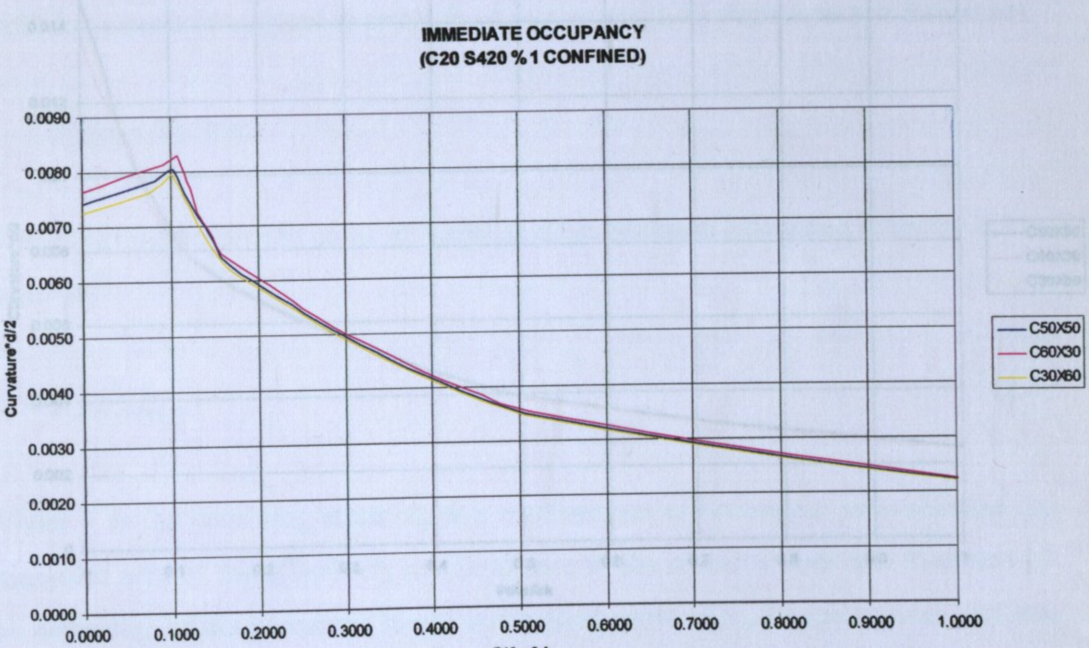


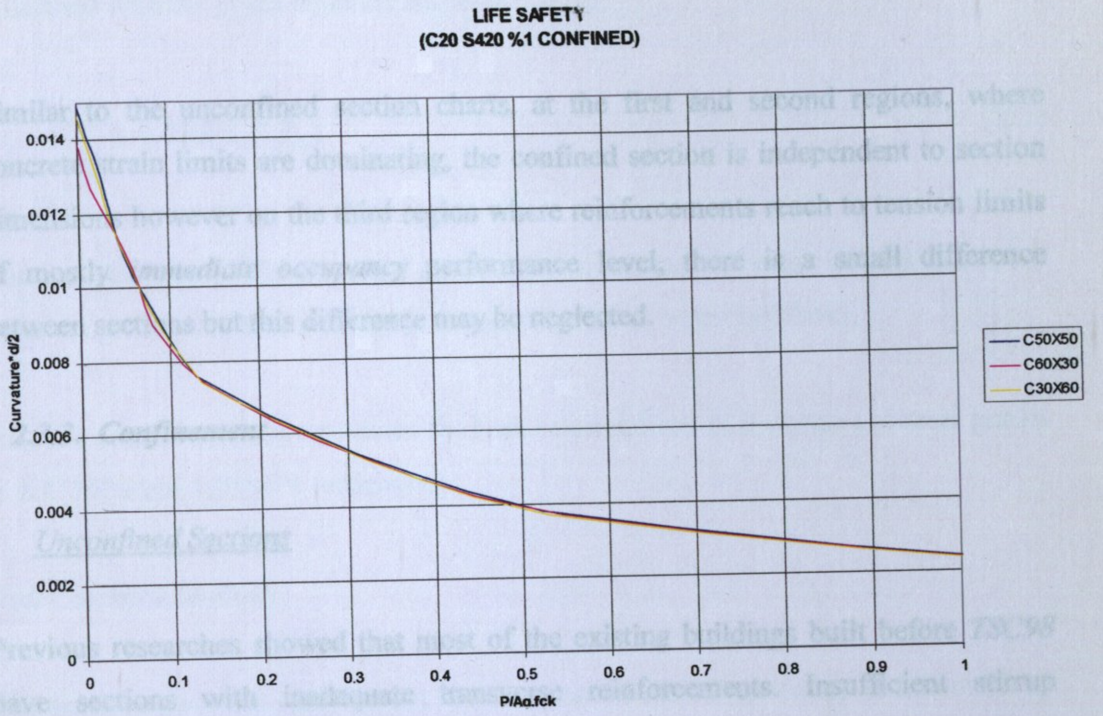
Figure 2.7. Effects of section dimensions for unconfined sections

At the first and second regions, where concrete strain limits are dominating, the unconfined section is independent to section dimensions however on the third region where reinforcements reach to tension limits of mostly *immediate occupancy* performance level, there is a small capacity difference between sections but this difference may be neglected.

ii. ***Confined Section***

In order to check whether the performance evaluation charts are dependent to the confined section dimensions or not, three analyses have been executed with three different sections "50^{cm}x50^{cm}", "30^{cm}x60^{cm}", "60^{cm}x30^{cm}". The comparison charts are shown below for three performance levels.





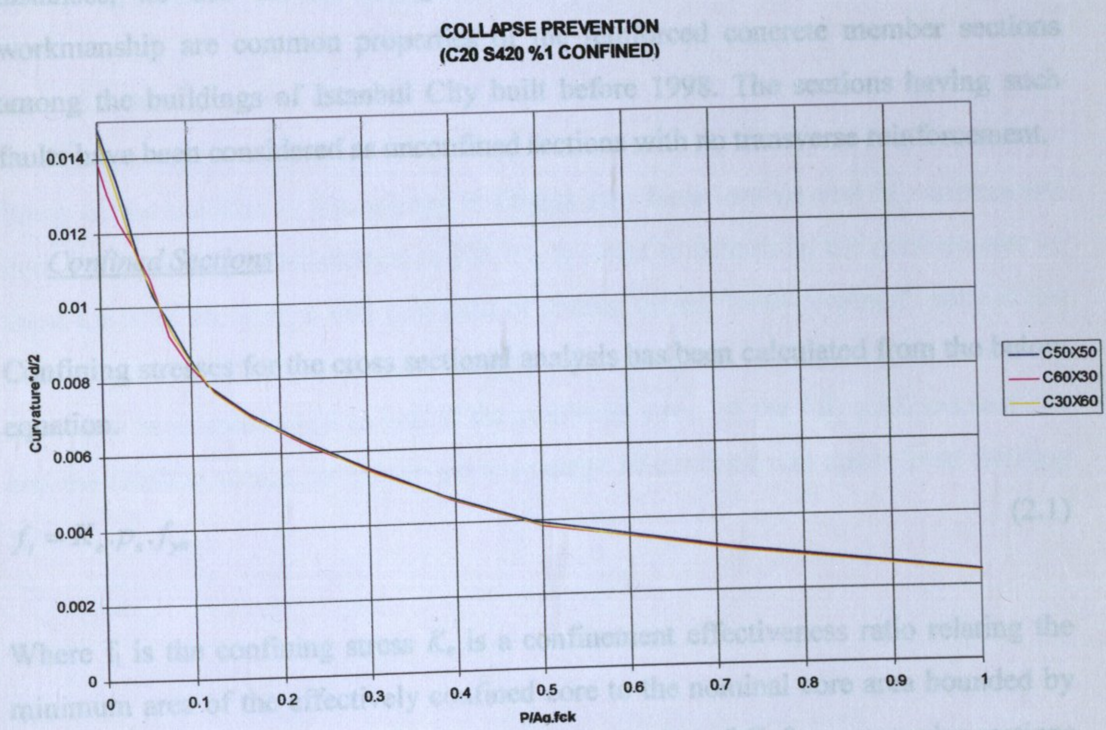


Figure 2.8. Effects of section dimensions for confined sections

Similar to the unconfined section charts, at the first and second regions, where concrete strain limits are dominating, the confined section is independent to section dimensions however on the third region where reinforcements reach to tension limits of mostly *immediate occupancy* performance level, there is a small difference between sections but this difference may be neglected.

2.3.2. Confinement

i. Unconfined Sections

Previous researches showed that most of the existing buildings built before *TSC98* have sections with inadequate transverse reinforcements. Insufficient stirrup distances, no use of ties along the sections, 90^0 bent hooks, and unskilled workmanship are common properties of the reinforced concrete member sections among the buildings of Istanbul City built before 1998. The sections having such faults have been considered as unconfined sections with no transverse reinforcement.

ii. Confined Sections

Confining stresses for the cross sectional analysis has been calculated from the below equation.

$$f_l = K_e.\rho_s.f_{yh} \tag{2.1}$$

Where f_l is the confining stress K_e is a confinement effectiveness ratio relating the minimum area of the effectively confined core to the nominal core area bounded by the centerline of the peripheral hoops and typical value of K_e for rectangular sections is *0.75*. ρ_s is the volumetric transverse reinforcement ratio and f_{yh} is transverse reinforcement yield stress.

ρ_s defined with the given equation below in *TSC98.*

$$\rho_s = \frac{A_{sh}}{s.b_k} \geq 0.075(f_{ck} / f_{yh}) \tag{2.2}$$

For concrete grade *C20* and steel grade *S420*; minimum ρ_s value is 0.0035 and for concrete grade *C25* and steel grade *S420* minimum ρ_s value is 0.0045.

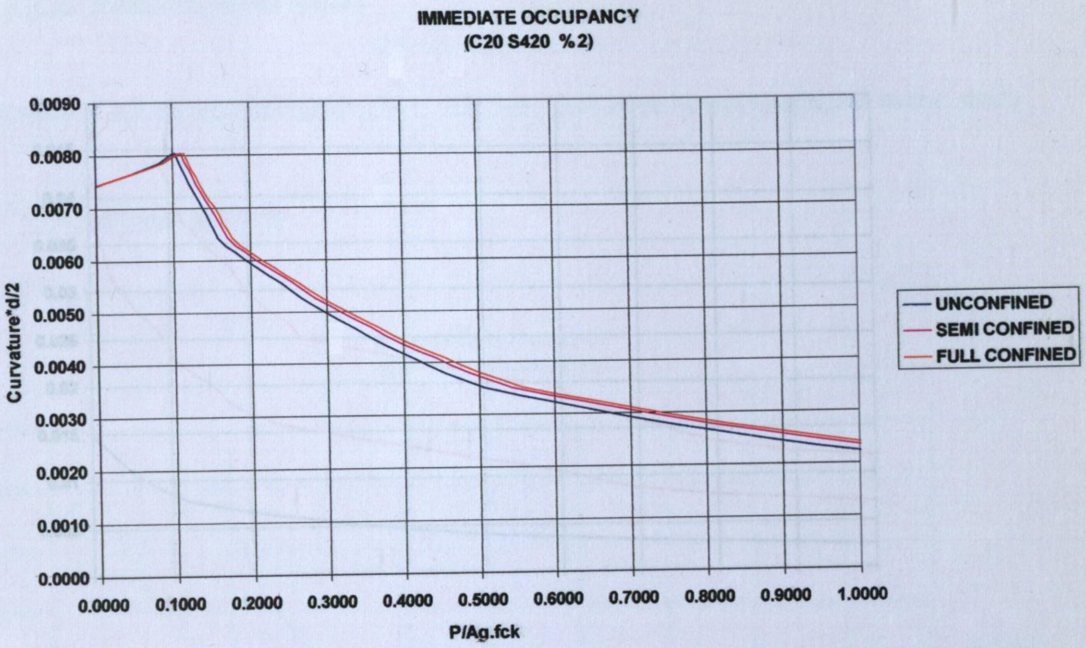
And the confining stress calculated for both combinations of concrete and steel grades is 1.47MPa and 1.89MPa respectively.

a) Full Confined Sections

The sections ensuring confinement requirements of *TSC98* are called as full confined (confined) sections with the confining stresses described above.

b) Semi Confined Sections

Some of the sections in the existing buildings may have less amount of confinement than the limit that is determined in *TSC98*. In order to determine the performance of these kinds of sections, a different kind of section called "*semi-confined*" section has been determined. In cross section analysis of these kinds of sections, the confining stress has been considered as half of the confining stress of the full confined sections and the limiting strains have been set to average of confined and unconfined limiting strains.


IMMEDIATE OCCUPANCY
(C20 S420 %2)
Curvature*d/2
0.0090
0.0080
0.0070
0.0060
0.0050
0.0040
0.0030
0.0020
0.0010
0.0000
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/Ag.fck
UNCONFINED
SEMI CONFINED
FULL CONFINED


LIFE SAFETY
(C20 S420 %2)
Curvature*d/2
0.035
0.03
0.025
0.02
0.015
0.01
0.005
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
P/Ag.fck
UNCONFINED
SEMI CONFINED
FULL CONFINED

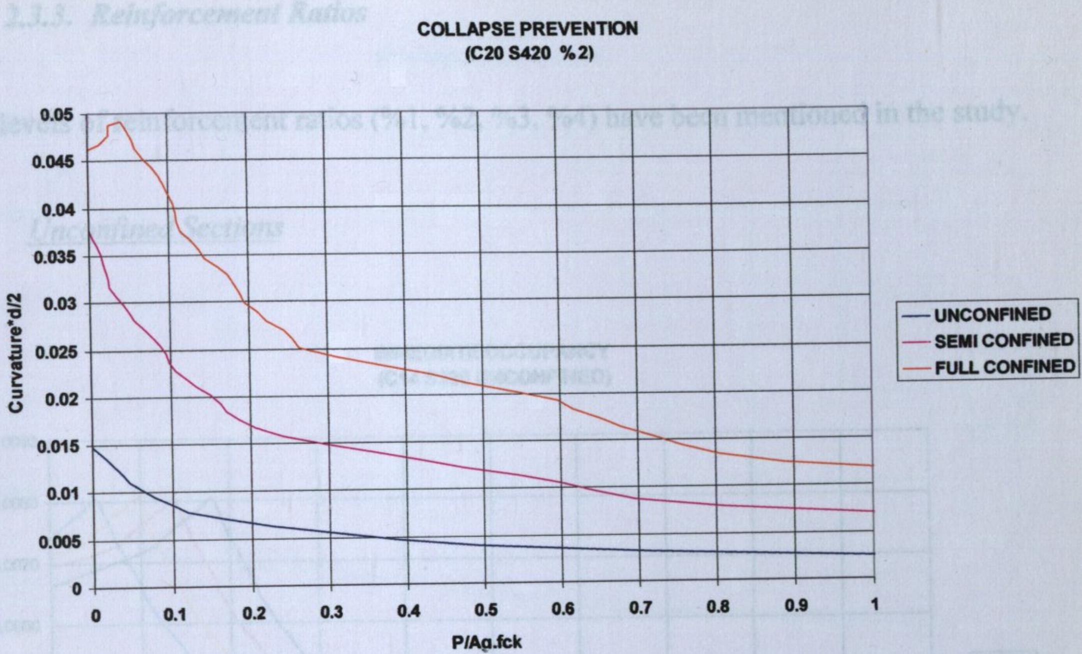


Figure 2.9. Effects of confinement levels

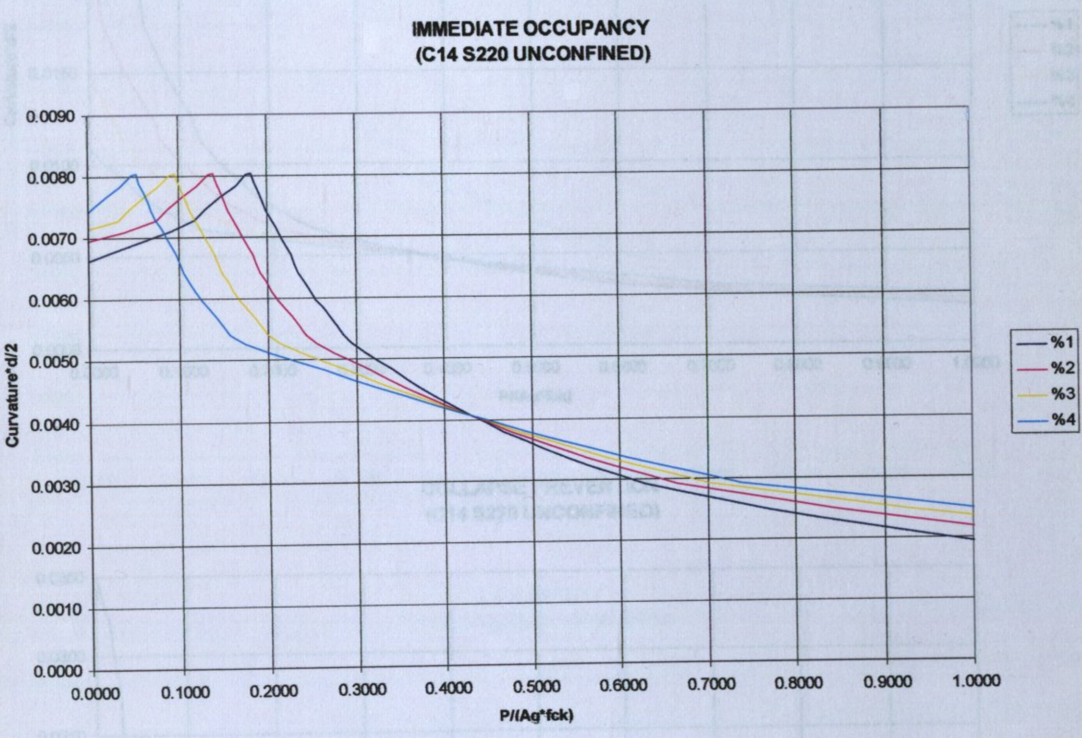
In the for *immediate occupancy* performance level represented above there is almost no difference between confined and unconfined sections because the limiting concrete strain of *immediate occupancy* performance level is totally relevant with the unconfined cover material.

The large plastic rotation capacity difference between unconfined, confined and semi confined sections is seen in the life safety and collapse prevention performance level charts due to the differences in confining stresses and limiting strains.

2.3.3. Reinforcement Ratios

Four levels of reinforcement ratios (%1, %2, %3, %4) have been mentioned in the study.

i. *Unconfined Sections*



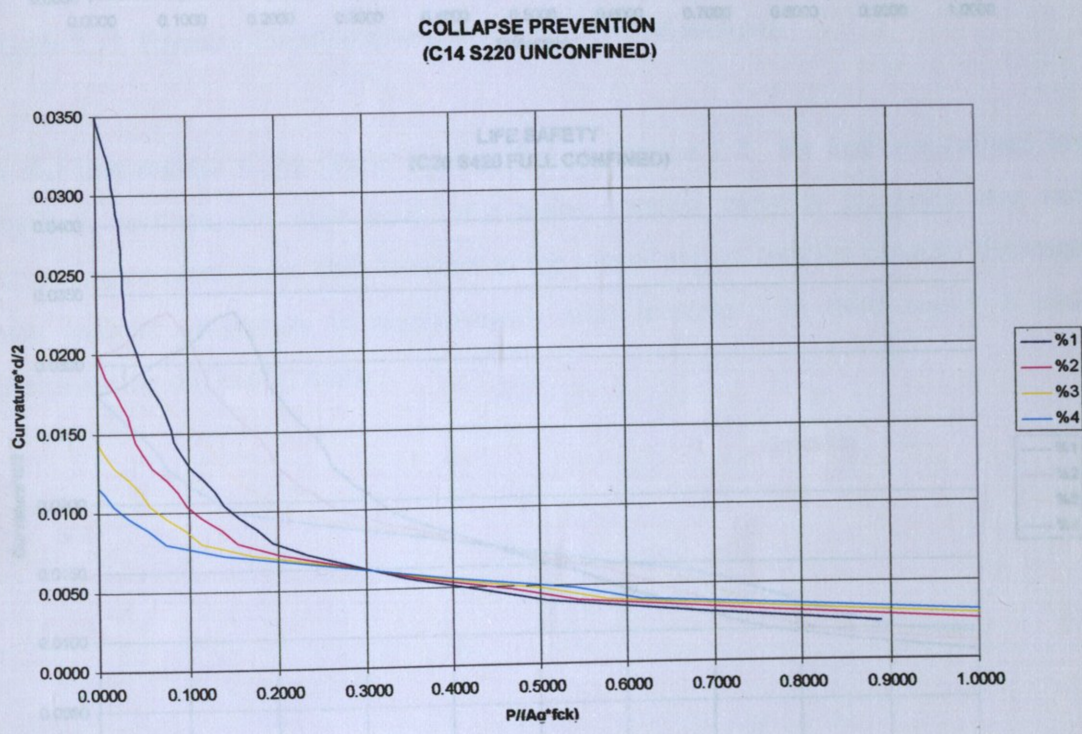
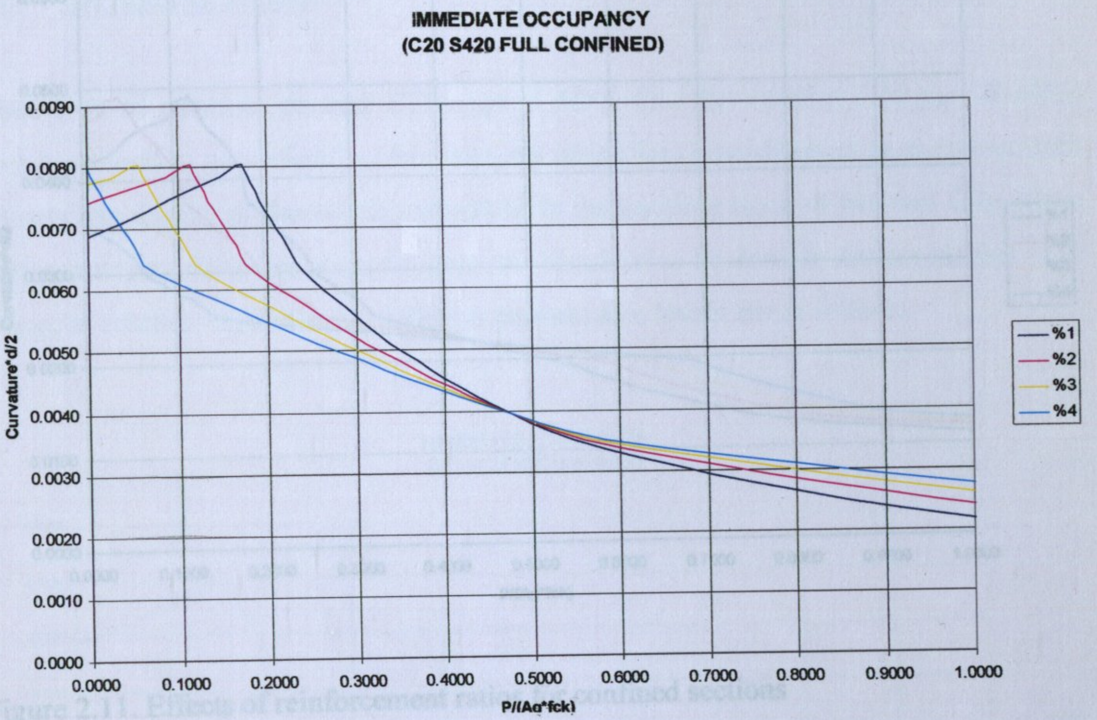
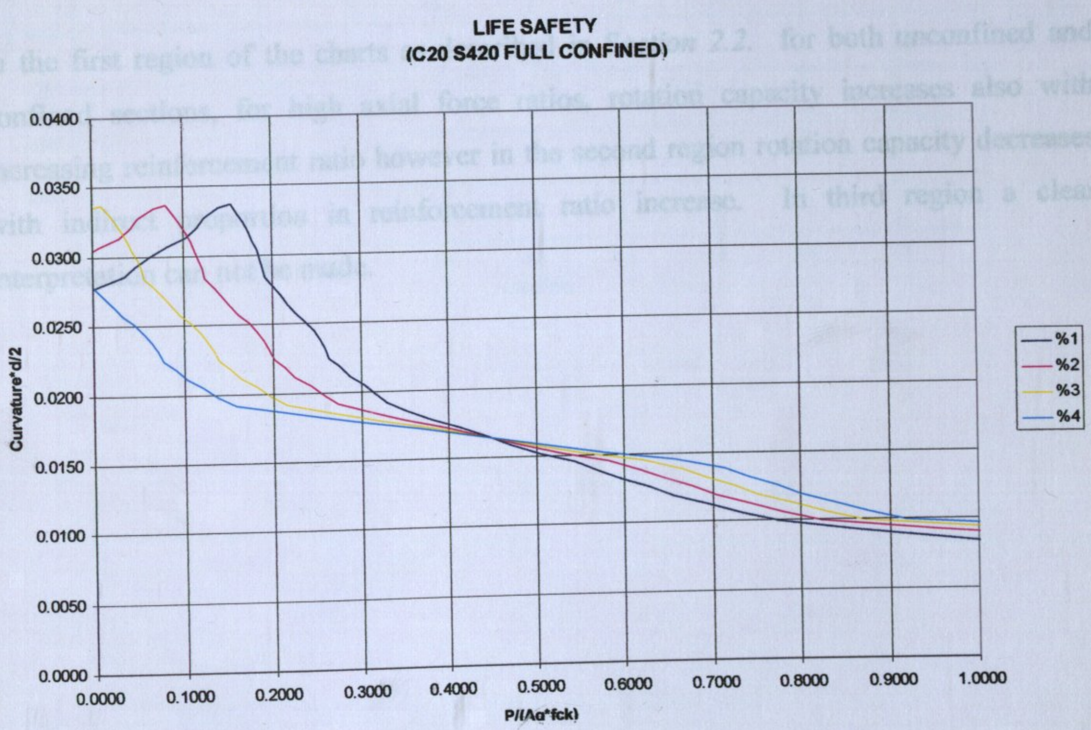


Figure 2.10. Effects of reinforcement ratios for unconfined sections

ii. Confined Sections





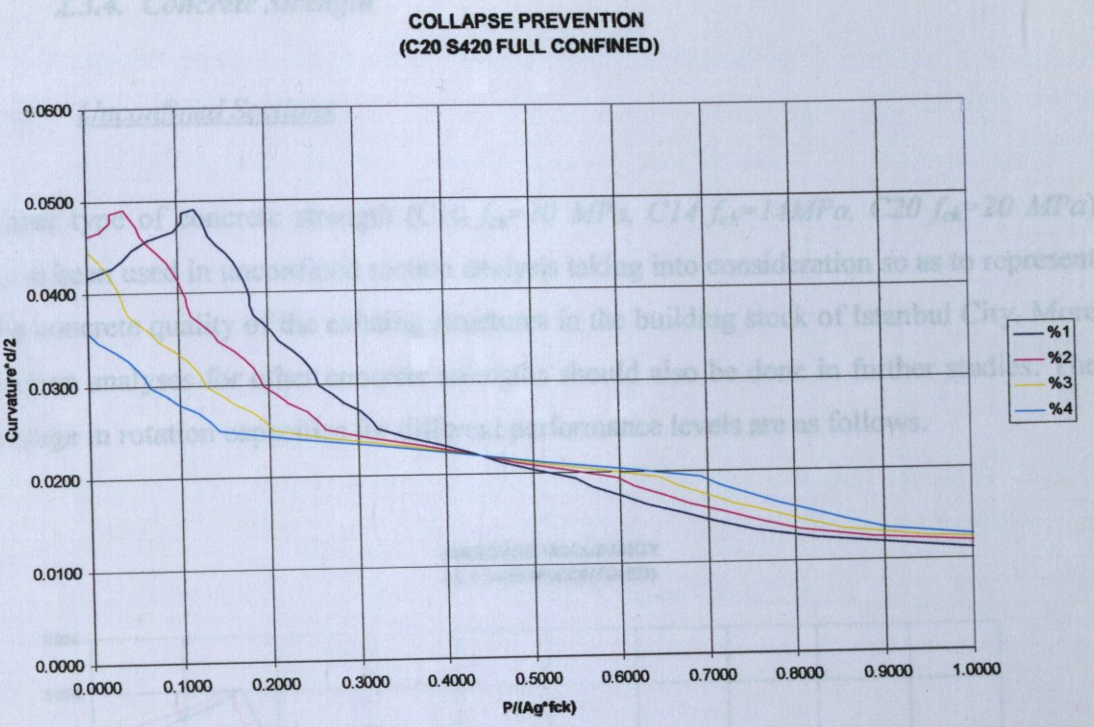


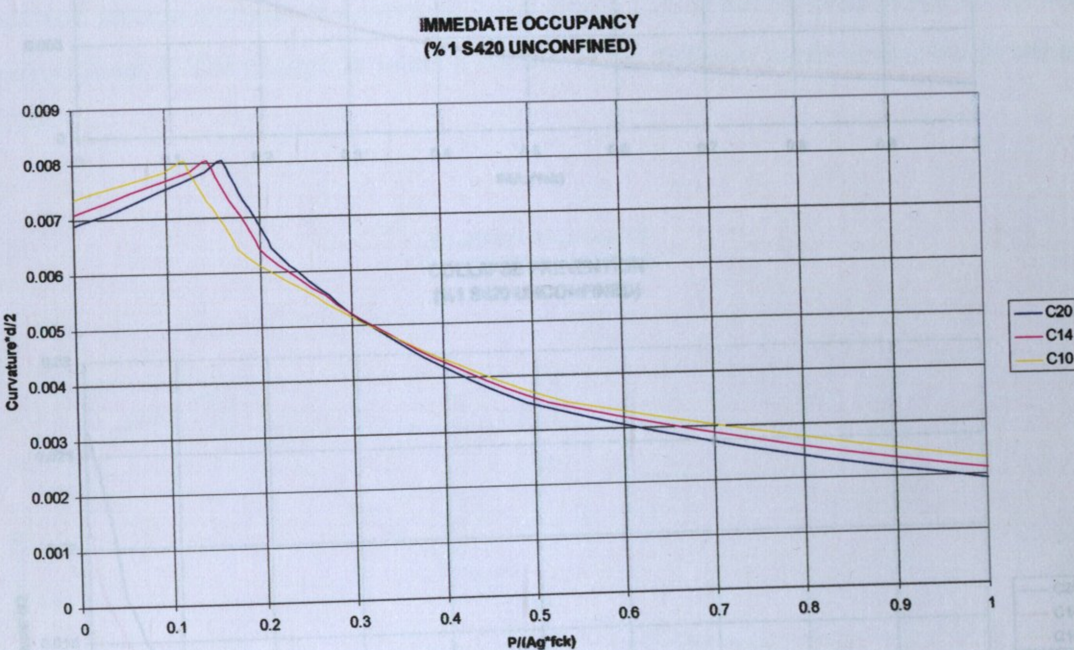
Figure 2.11. Effects of reinforcement ratios for confined sections

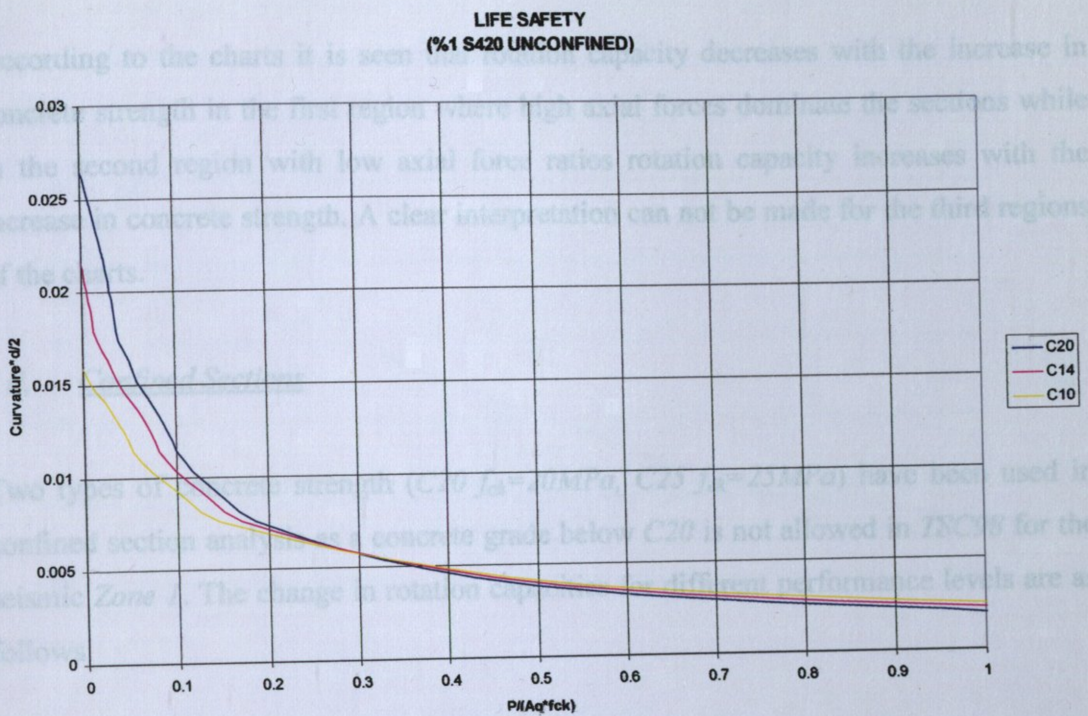
In the first region of the charts as described in S*ection 2.2.* for both unconfined and confined sections, for high axial force ratios, rotation capacity increases also with increasing reinforcement ratio however in the second region rotation capacity decreases with indirect proportion in reinforcement ratio increase. In third region a clear interpretation can not be made.

2.3.4. *Concrete Strength*

i. Unconfined Sections

Three type of concrete strength (C10 f_{ck}=*10 MPa, C14* f_{ck}=*14MPa, C20* f_{ck}=*20 MPa*) have been used in unconfined section analysis taking into consideration so as to represent the concrete quality of the existing structures in the building stock of Istanbul City. More section analyses for other concrete strengths should also be done in further studies. The change in rotation capacities for different performance levels are as follows.





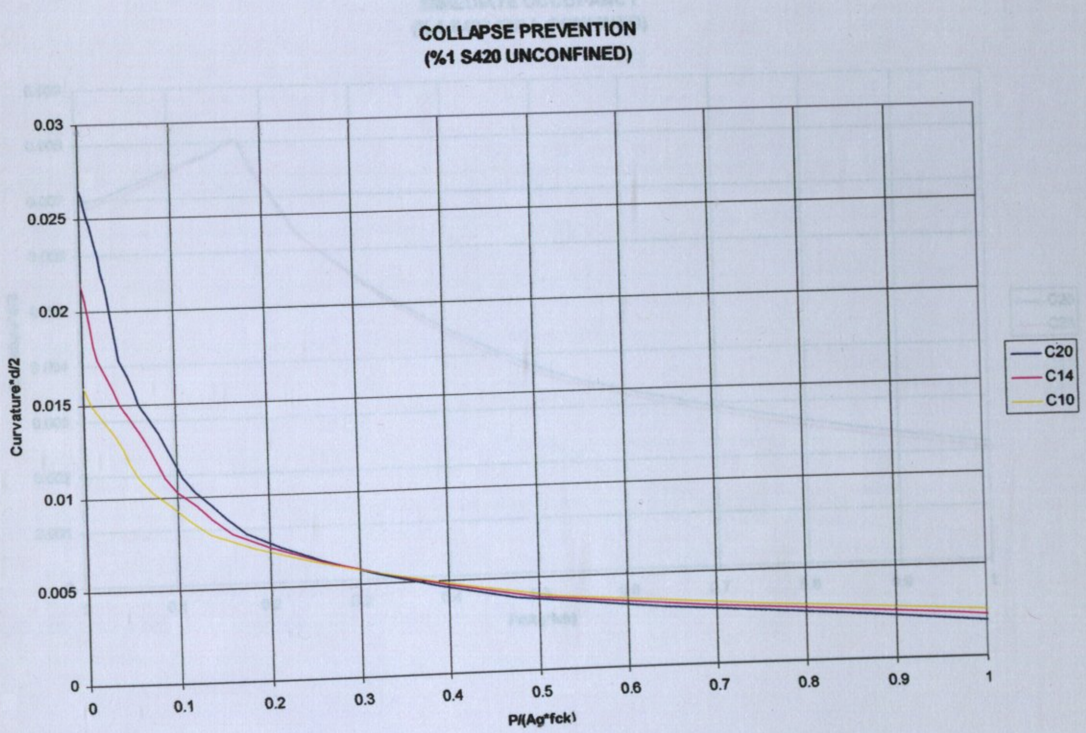
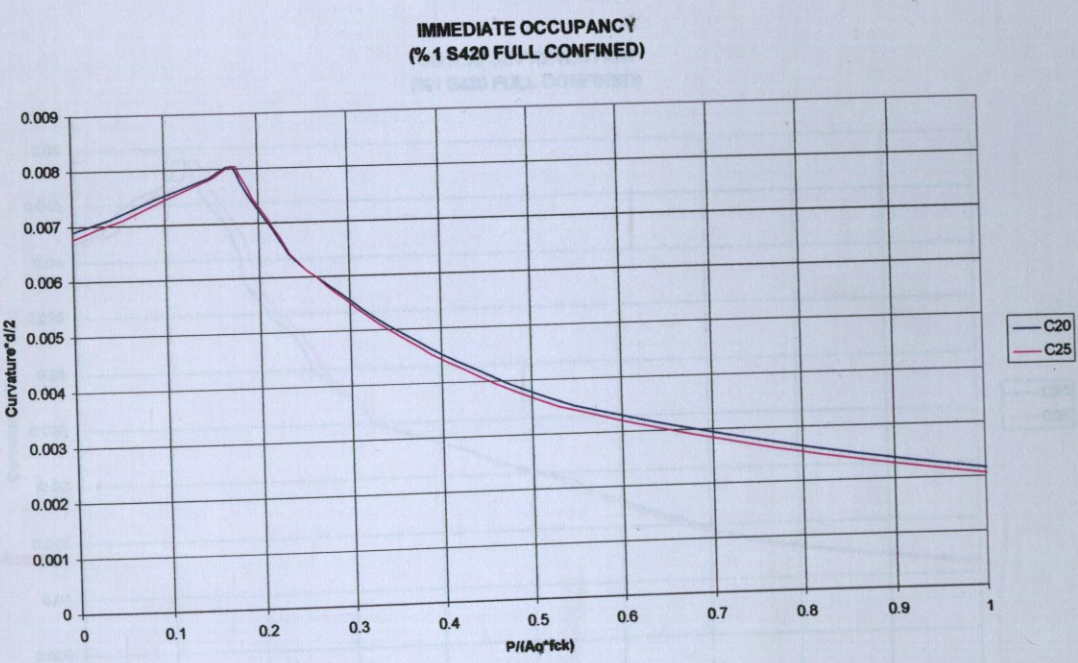


Figure 2.12. Effects of concrete grades for unconfined sections

According to the charts it is seen that rotation capacity decreases with the increase in concrete strength in the first region where high axial forces dominate the sections while in the second region with low axial force ratios rotation capacity increases with the increase in concrete strength. A clear interpretation can not be made for the third regions of the charts.

ii. *Confined Sections*

Two types of concrete strength (*C20* f_{ck}*=20MPa, C25* f_{ck}*=25MPa*) have been used in confined section analysis as a concrete grade below *C20* is not allowed in *TSC98* for the seismic *Zone 1*. The change in rotation capacities for different performance levels are as follows.



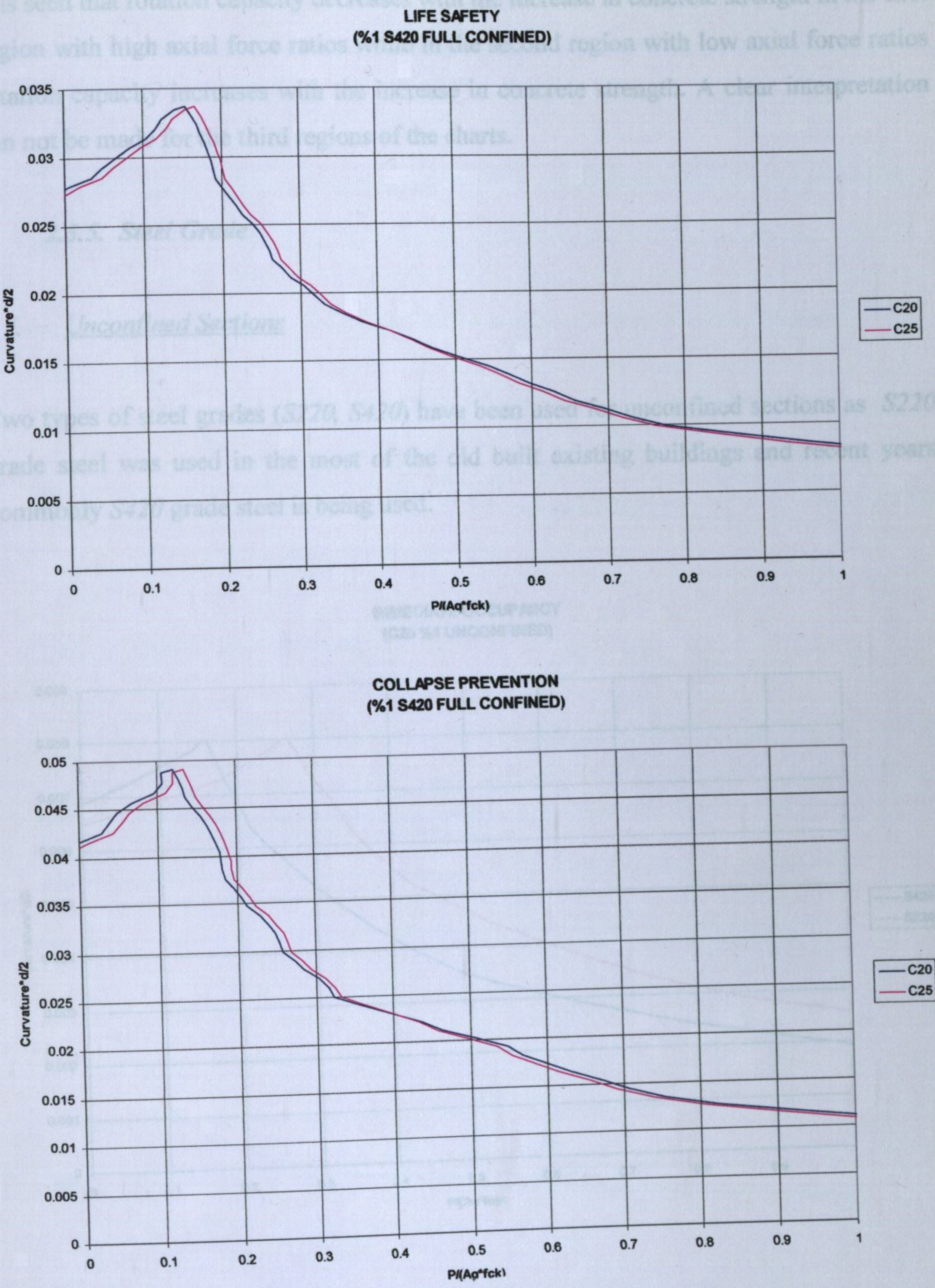


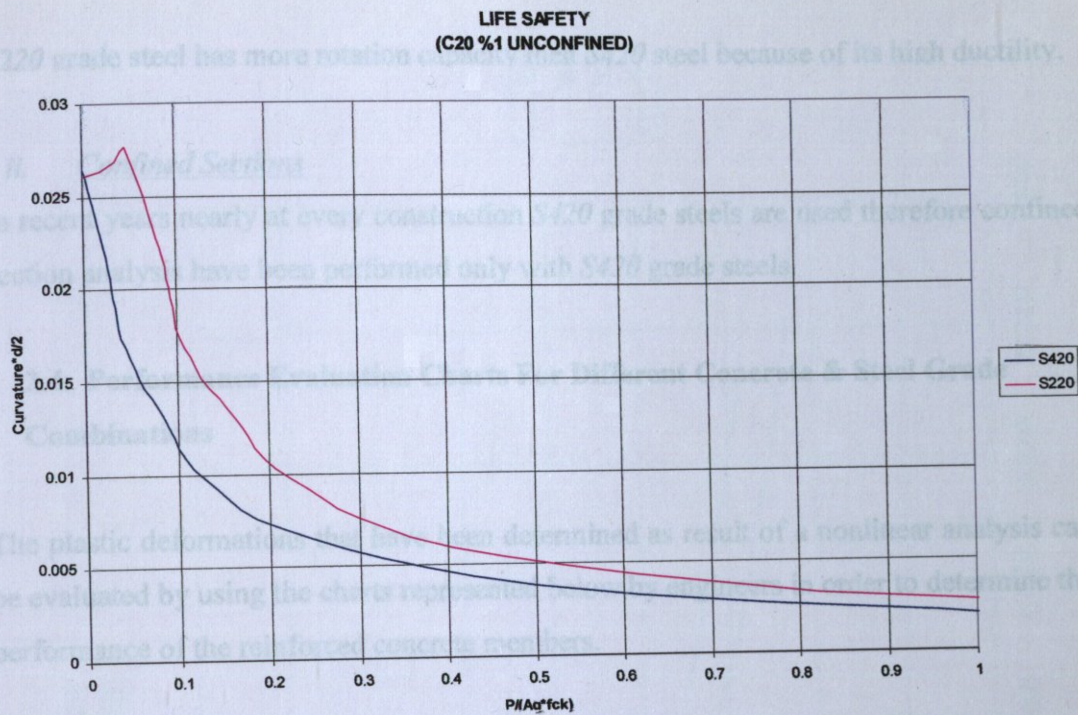
Figure 2.13. Effects of concrete grades for confined sections

It is seen that rotation capacity decreases with the increase in concrete strength in the first region with high axial force ratios while in the second region with low axial force ratios rotation capacity increases with the increase in concrete strength. A clear interpretation can not be made for the third regions of the charts.

2.3.5. Steel Grade

i. *Unconfined Sections*

Two types of steel grades (*S220, S420*) have been used for unconfined sections as *S220* grade steel was used in the most of the old built existing buildings and recent years commonly *S420* grade steel is being used.



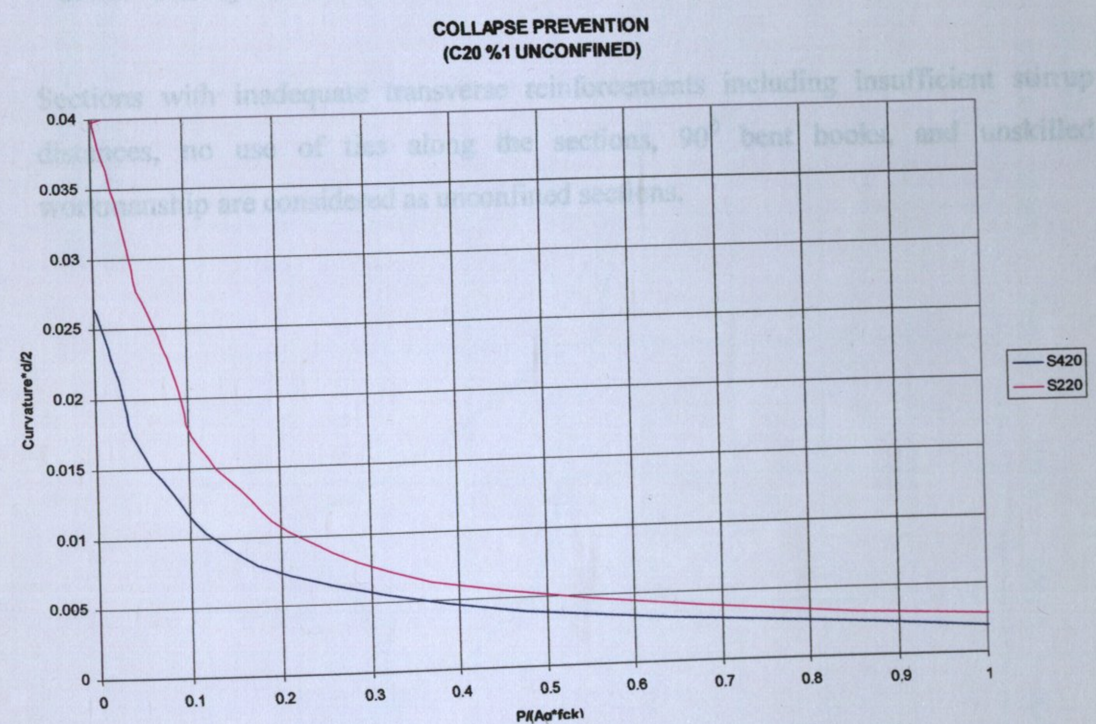


Figure 2.14. Effects of reinforcing steel grades for unconfined sections

S220 grade steel has more rotation capacity then *S420* steel because of its high ductility.

ii. Confined Sections

In recent years nearly at every construction *S420* grade steels are used therefore confined section analysis have been performed only with *S420* grade steels.

2.4. Performance Evaluation Charts For Different Concrete & Steel Grade Combinations

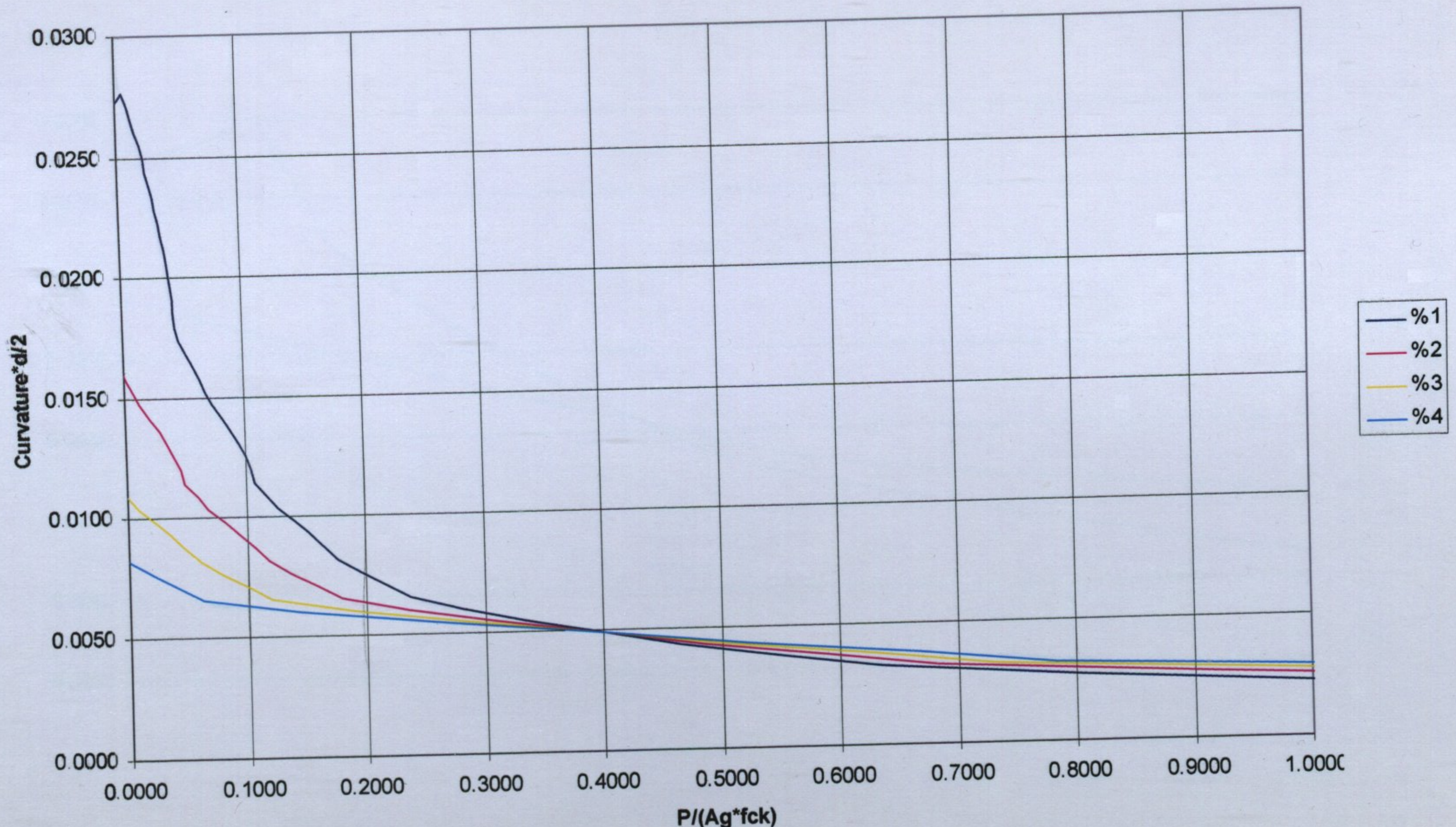
The plastic deformations that have been determined as result of a nonlinear analysis can be evaluated by using the charts represented below by engineers in order to determine the performance of the reinforced concrete members.

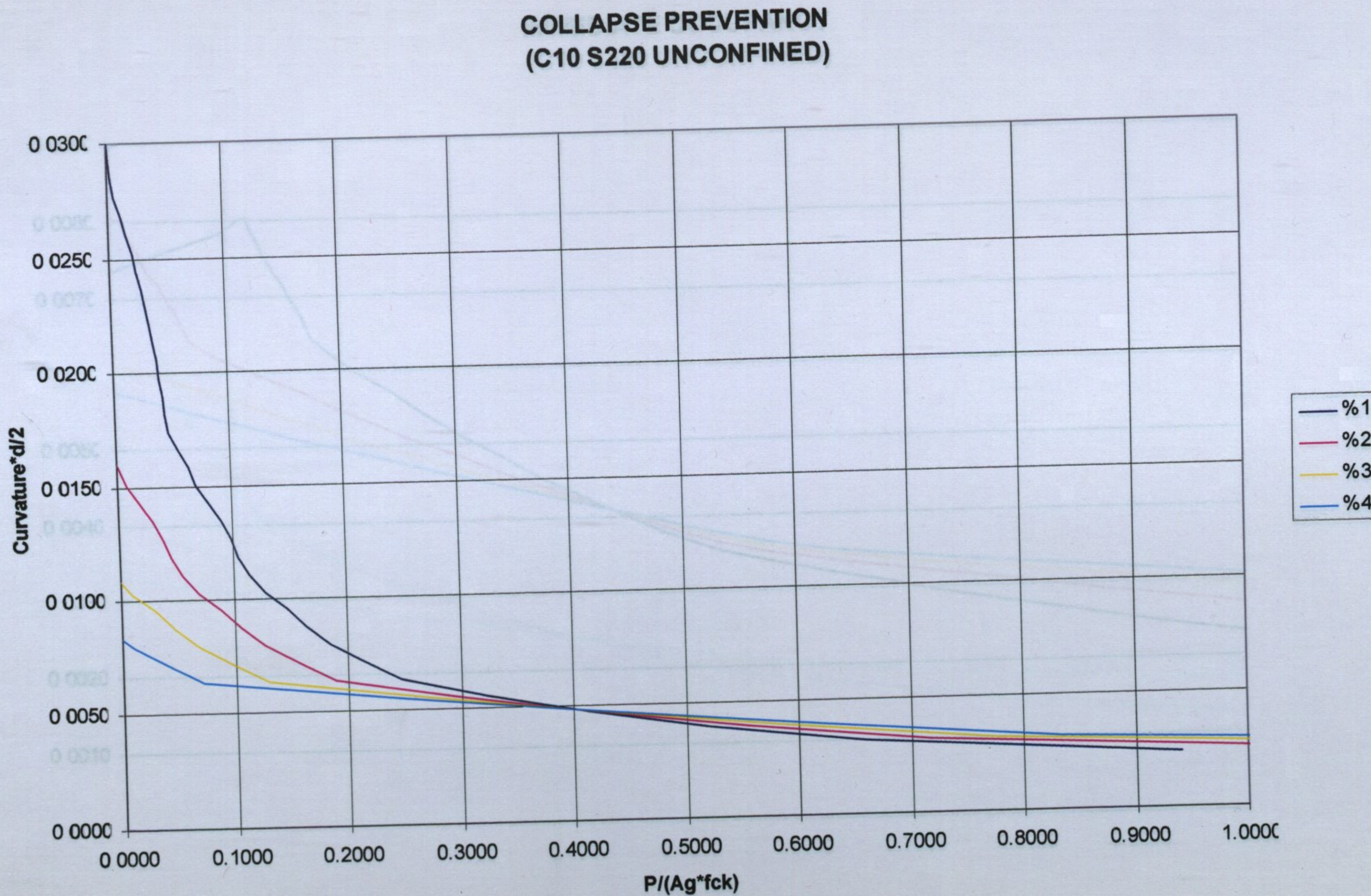
2.4.1. Unconfined Sections

Sections with inadequate transverse reinforcements including insufficient stirrup distances, no use of ties along the sections, 90^0 bent hooks, and unskilled workmanship are considered as unconfined sections.

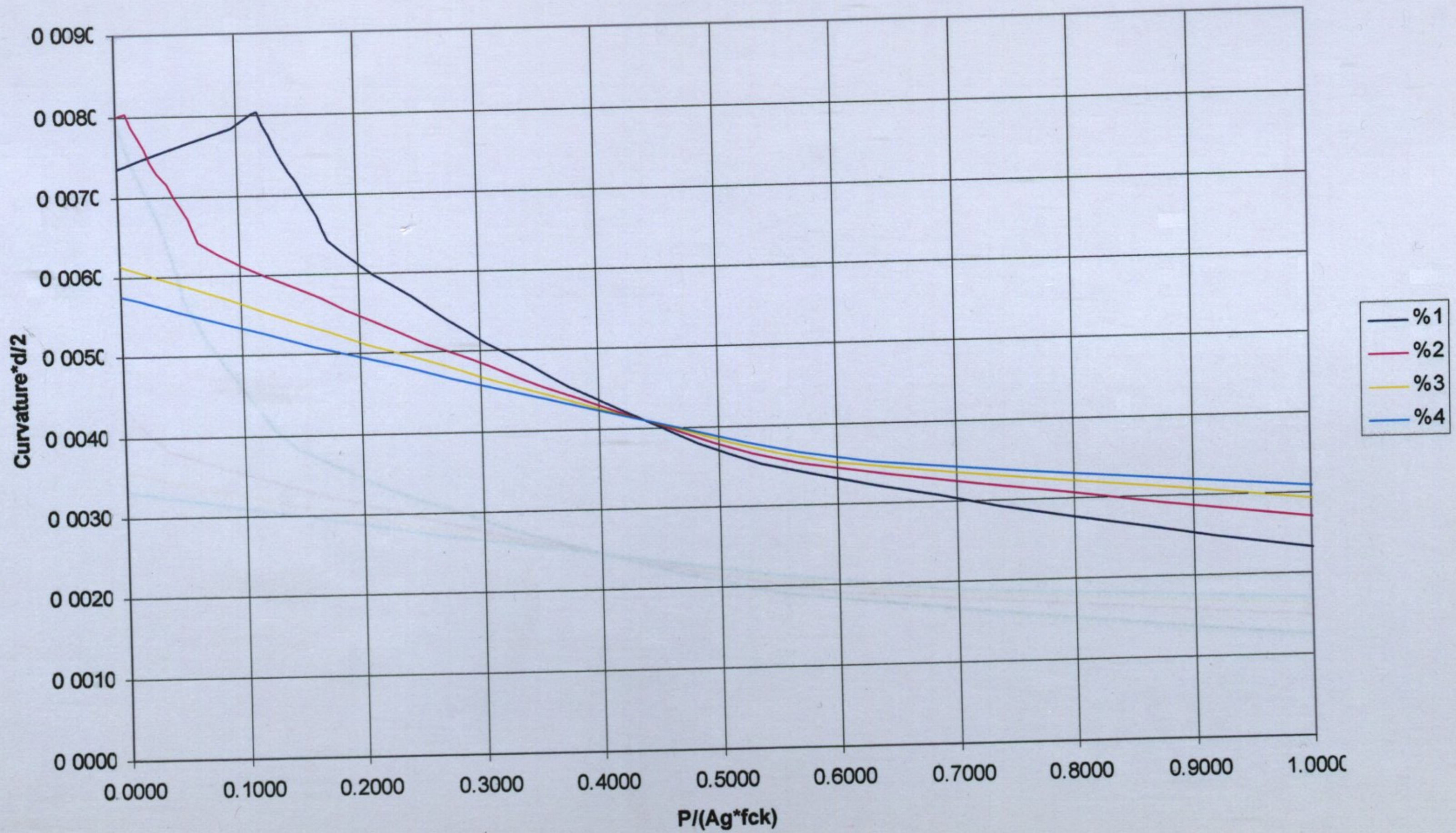

IMMEDIATE OCCUPANCY
(C10 S220 UNCONFINED)
0 0090
0 0080
0 0070
0 0060
0 0050
0 0040
0 0030
0 0020
0 0010
0 0000
Curvature*d/2
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4

LIFE SAFETY
(C10 S220 UNCONFINED)

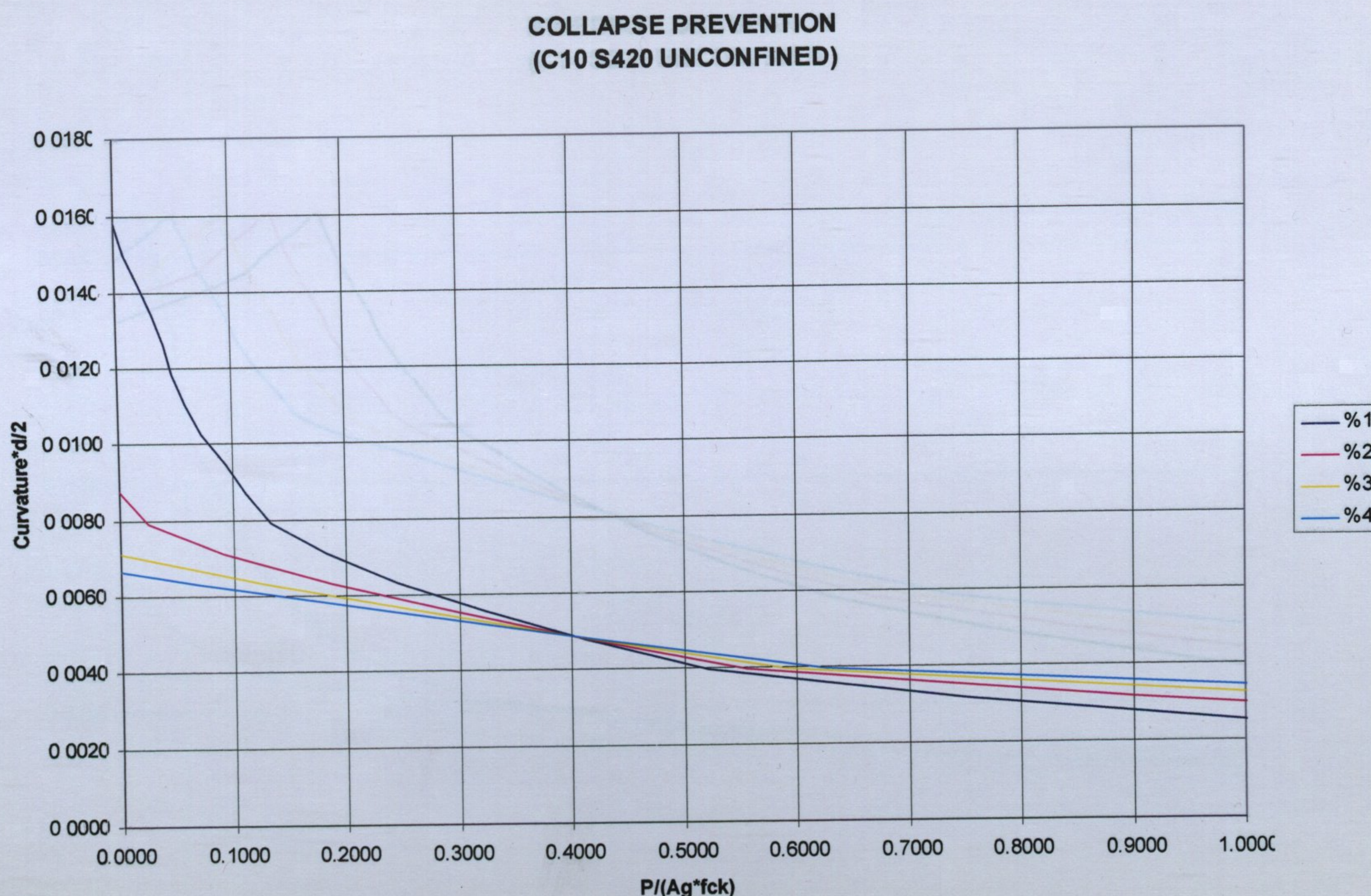


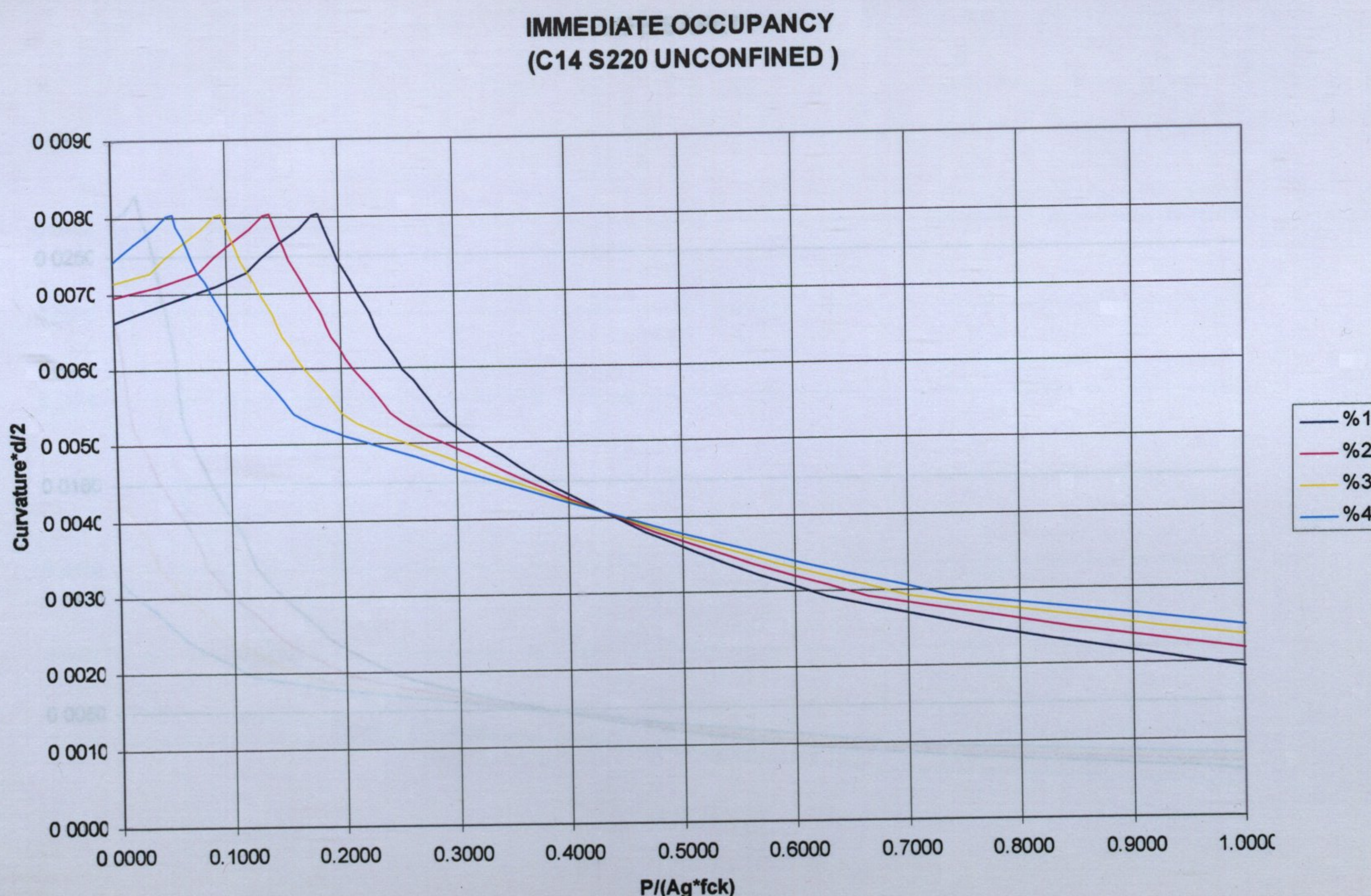

COLLAPSE PREVENTION
(C10 S220 UNCONFINED)
Curvature*d/2
0 0300
0 0250
0 0200
0 0150
0 0100
0 0050
0 0000
0 0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4

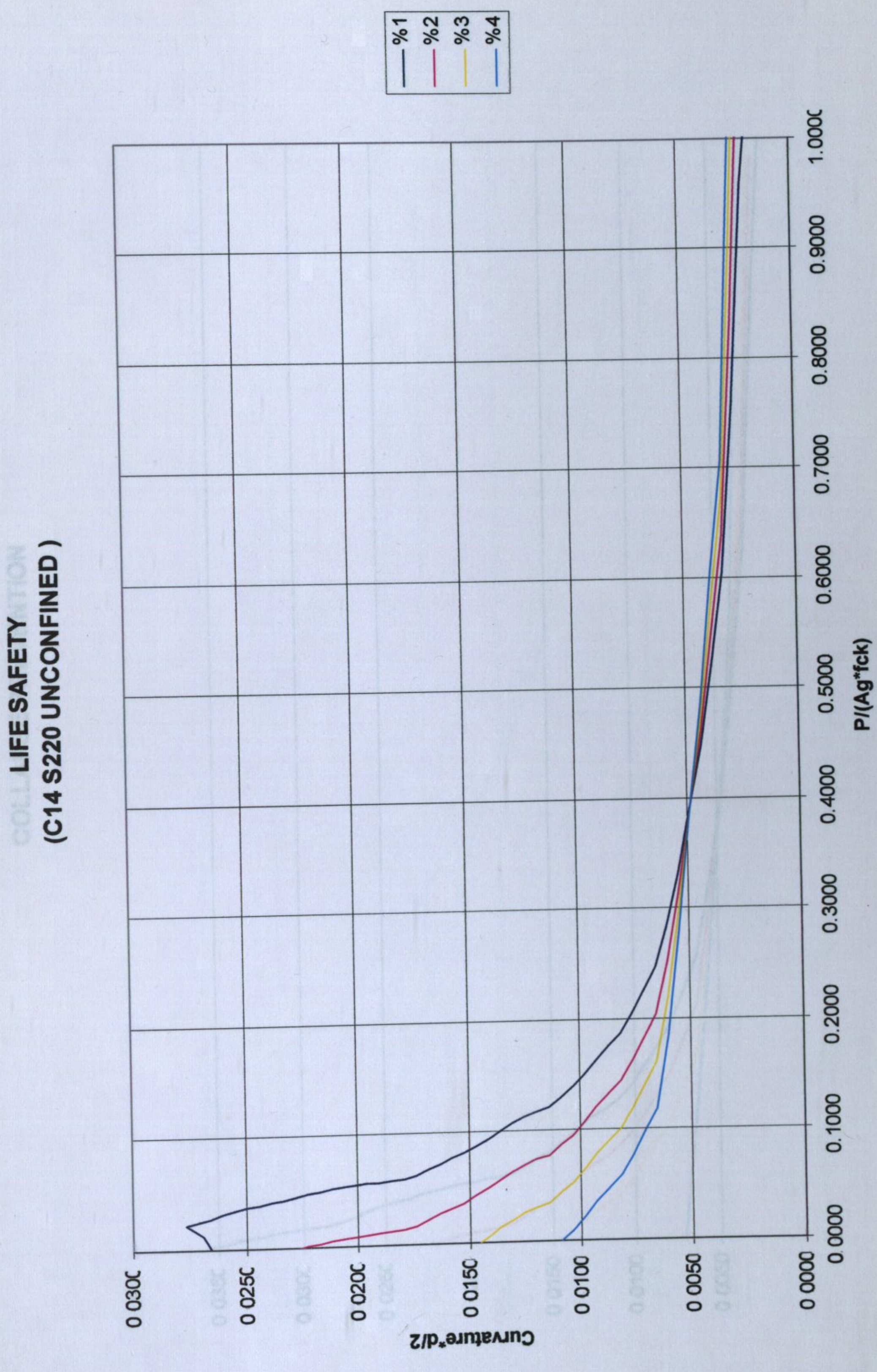
IMMEDIATE OCCUPANCY
(C10 S420 UNCONFINED)

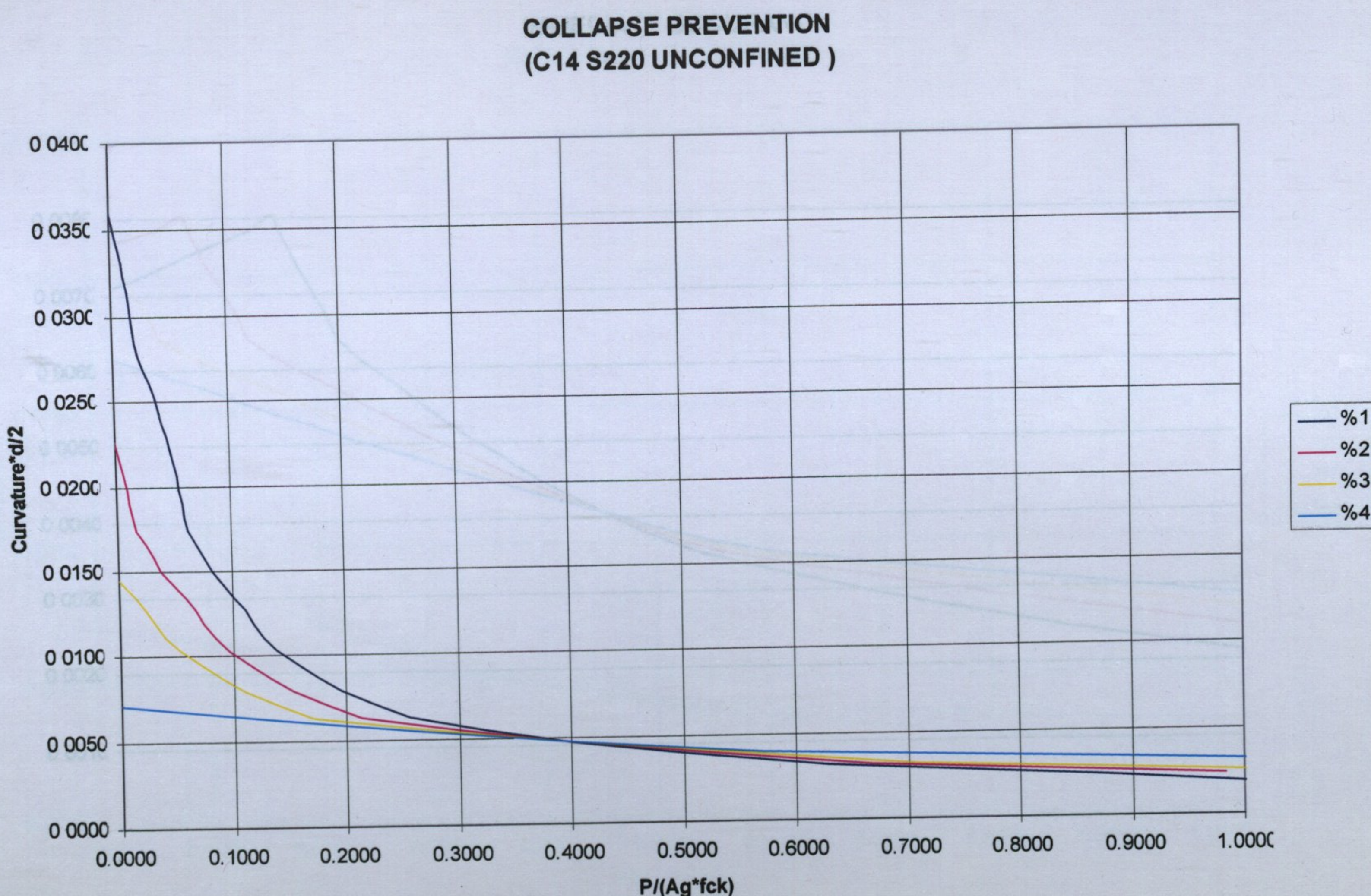


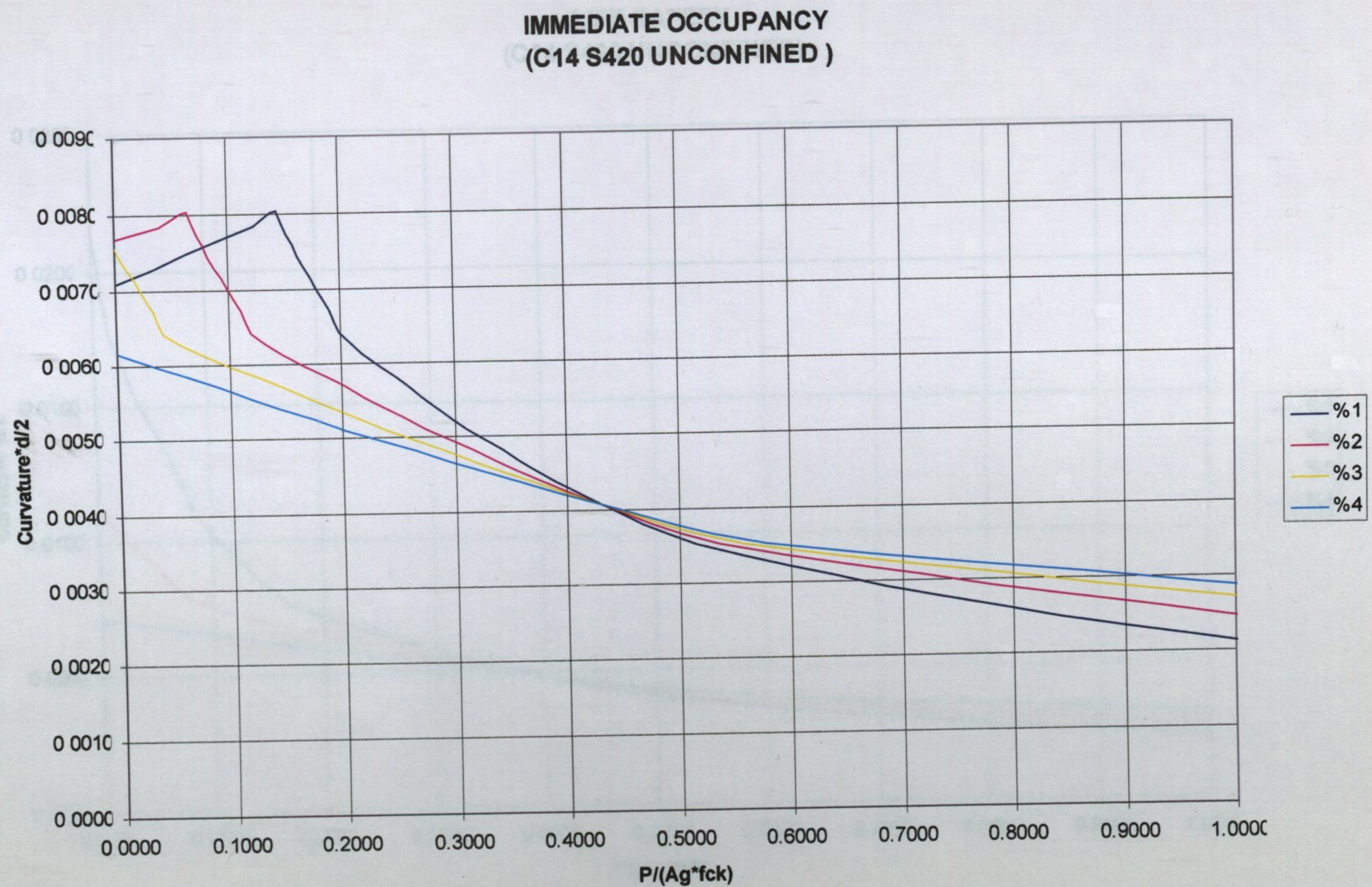

LIFE SAFETY
(C10 S420 UNCONFINED)
Curvature*d/2
P/(Ag*fck)
0 0000
0 0020
0 0040
0 0060
0 0080
0 0100
0 0120
0 0140
0 0160
0 0180
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
%1
%2
%3
%4


COLLAPSE PREVENTION
(C10 S420 UNCONFINED)
0 0180
0 0160
0 0140
0 0120
0 0100
0 0080
0 0060
0 0040
0 0020
0 0000
Curvature*d/2
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4

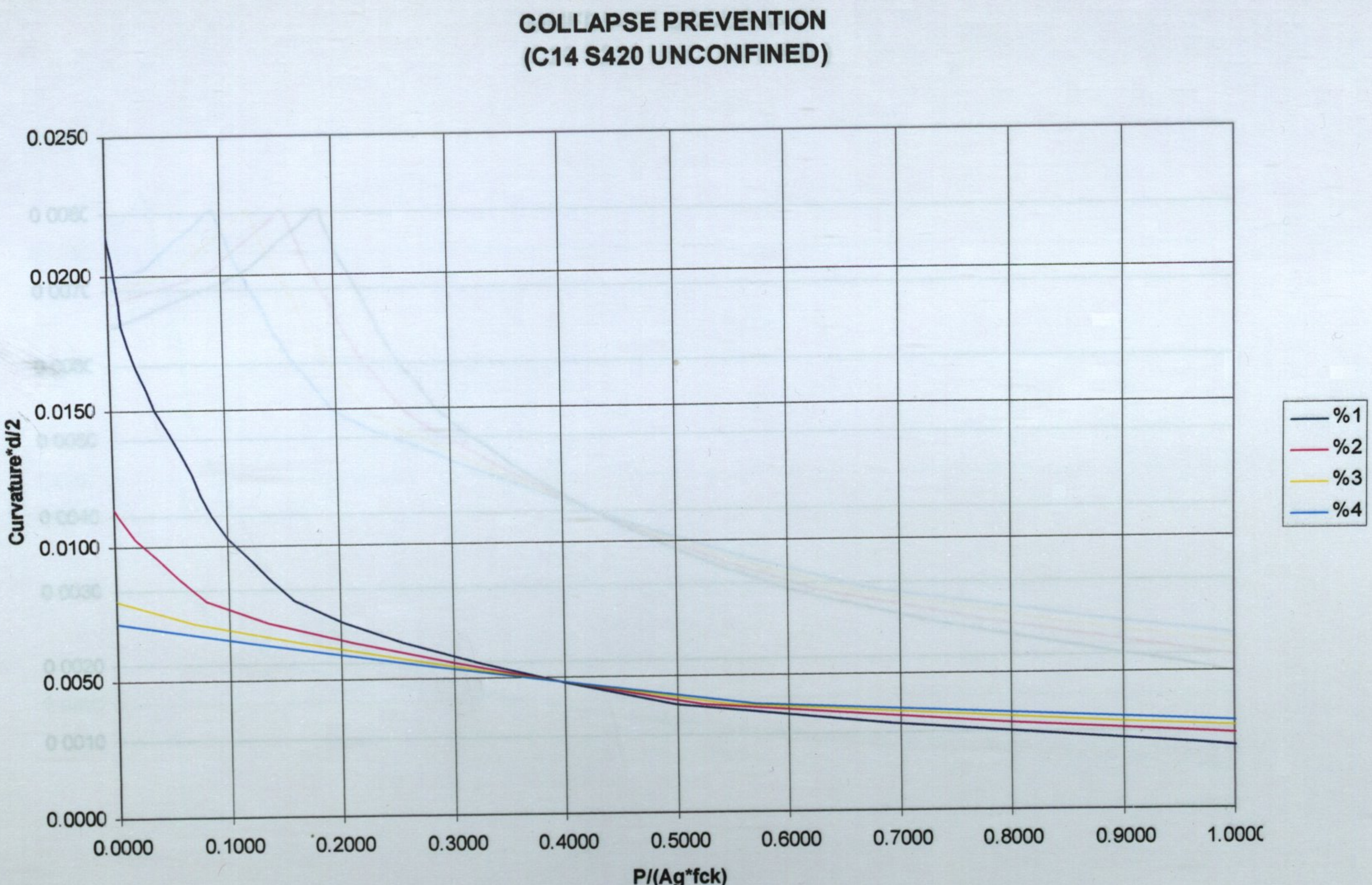

IMMEDIATE OCCUPANCY
(C14 S220 UNCONFINED)
Curvature*d/2
0 0090
0 0080
0 0070
0 0060
0 0050
0 0040
0 0030
0 0020
0 0010
0 0000
0 0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4


LIFE SAFETY
(C14 S220 UNCONFINED)
Curvature*d/2
P/(Ag*fck)
0.0000
0.0050
0.0100
0.0150
0.0200
0.0250
0.0300
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
%1
%2
%3
%4


COLLAPSE PREVENTION
(C14 S220 UNCONFINED)
0 0400
0 0350
0 0300
0 0250
0 0200
0 0150
0 0100
0 0050
0 0000
Curvature*d/2
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4

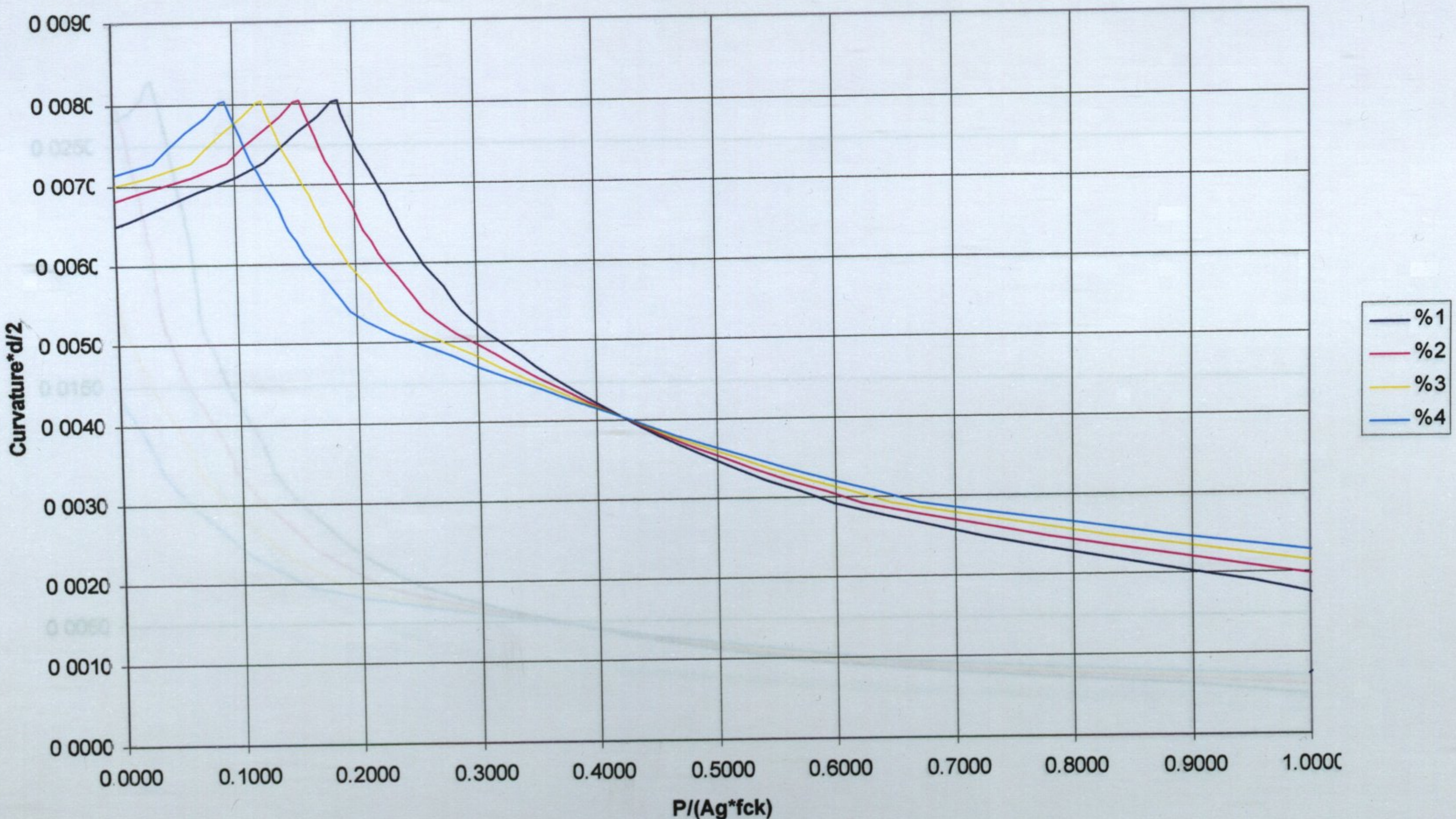

IMMEDIATE OCCUPANCY
(C14 S420 UNCONFINED)
Curvature*d/2
0 0090
0 0080
0 0070
0 0060
0 0050
0 0040
0 0030
0 0020
0 0010
0 0000
0 0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4

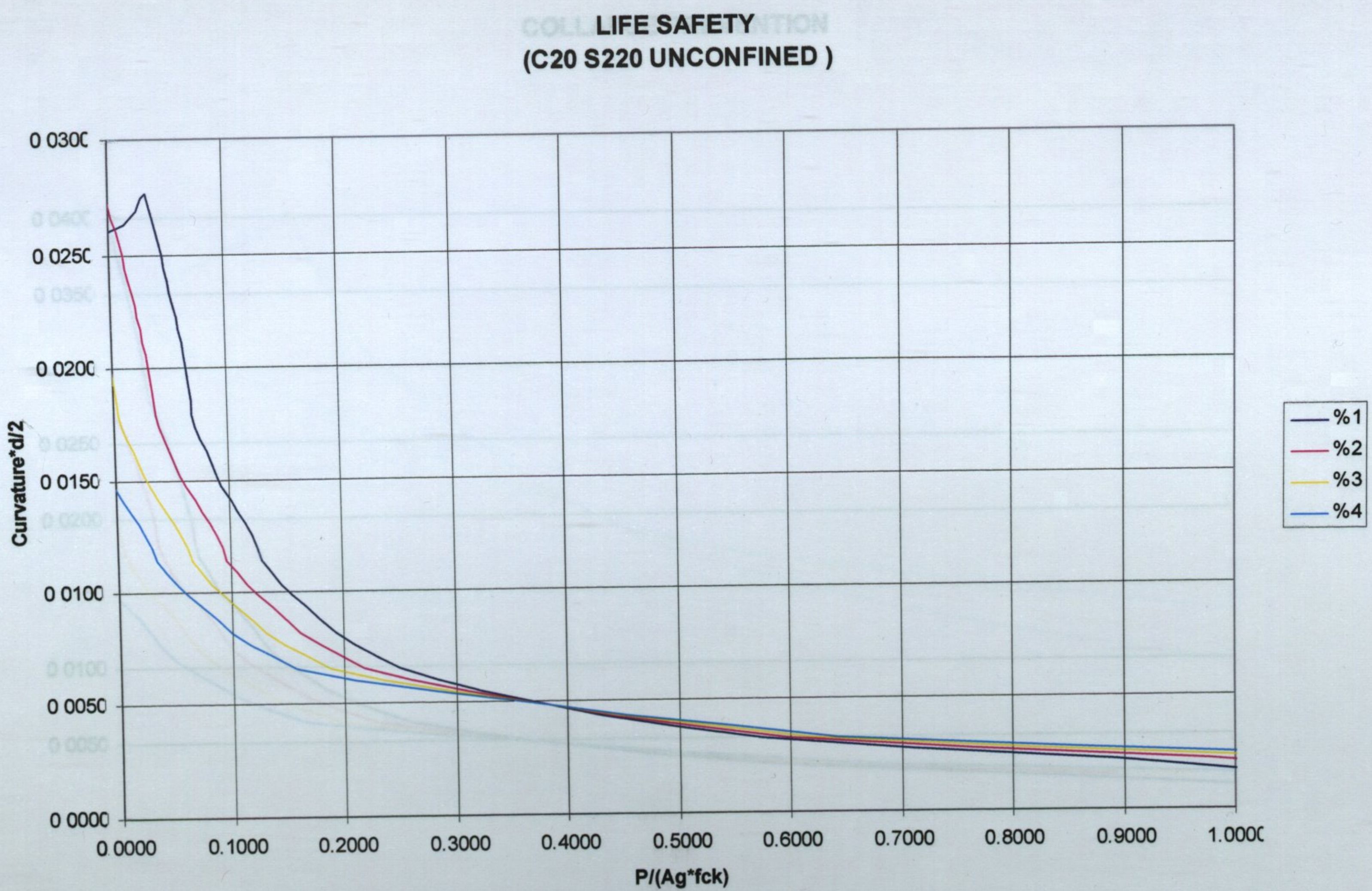
LIFE SAFETY
(C14 S420 UNCONFINED)
0 0250
0 0200
0 0150
0 0100
0 0050
0 0000
Curvature*d/2
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4

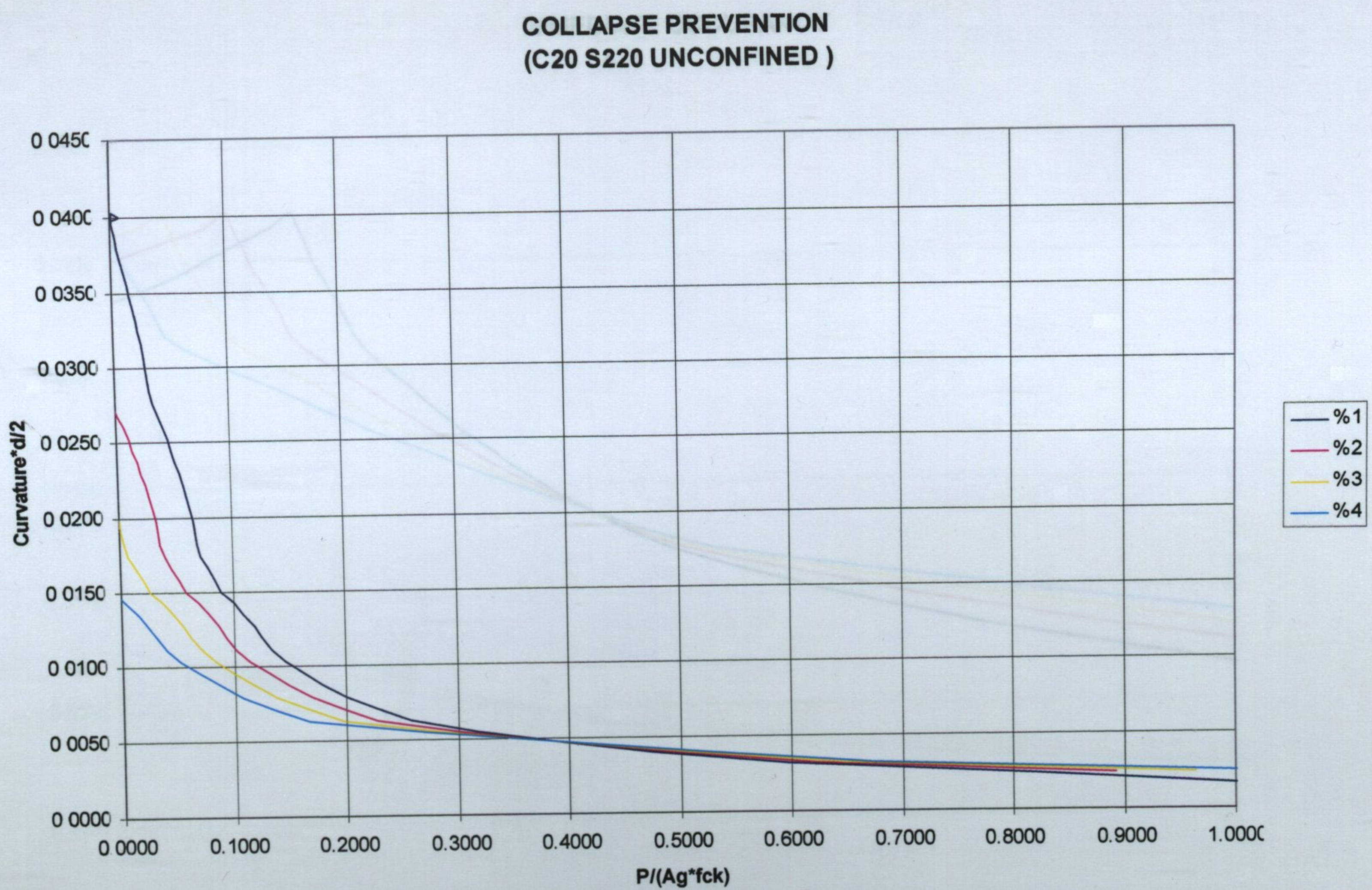


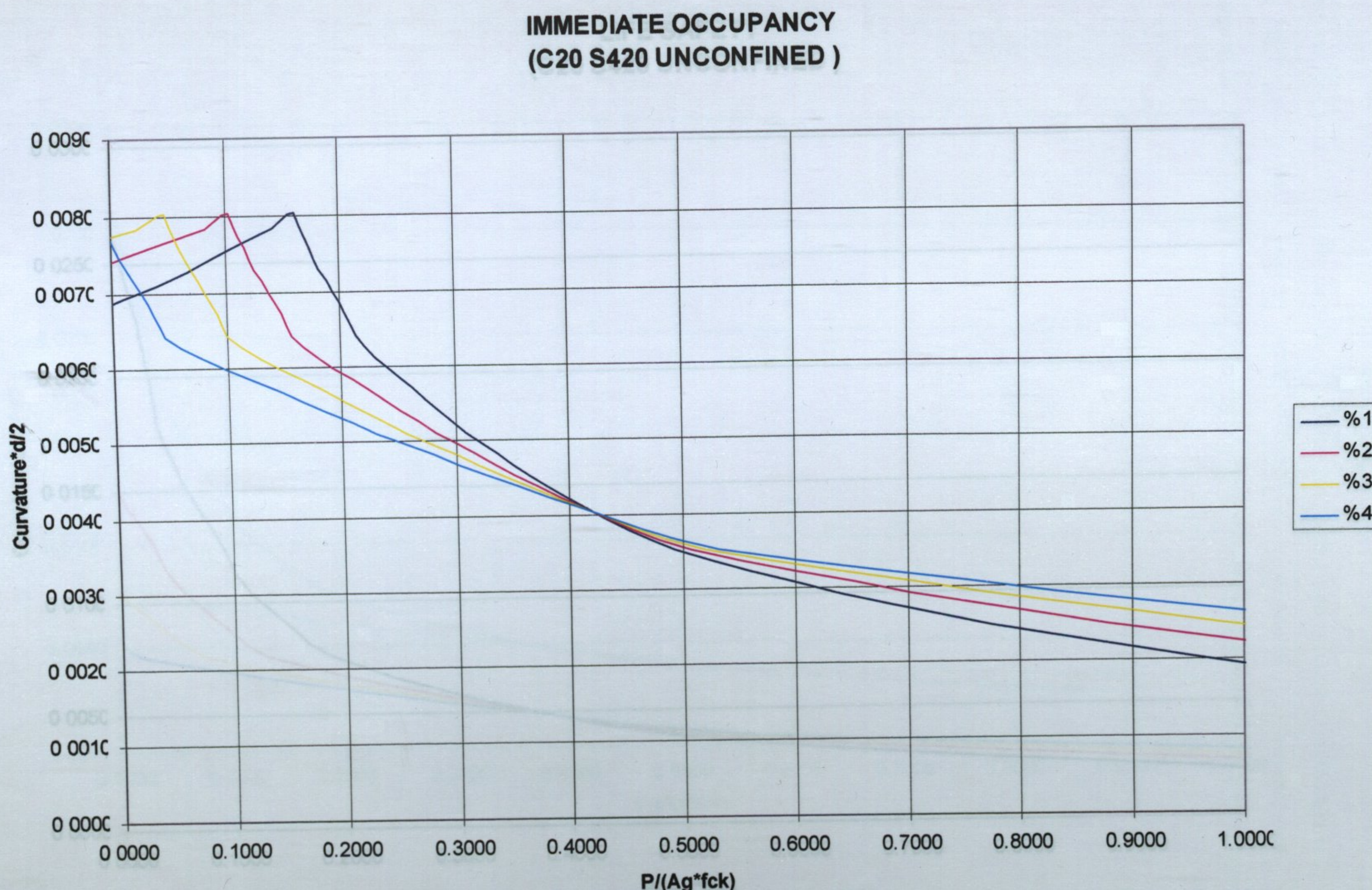
COLLAPSE PREVENTION
(C14 S420 UNCONFINED)
Curvature*d/2
0.0250
0.0200
0.0150
0.0100
0.0050
0.0000
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4

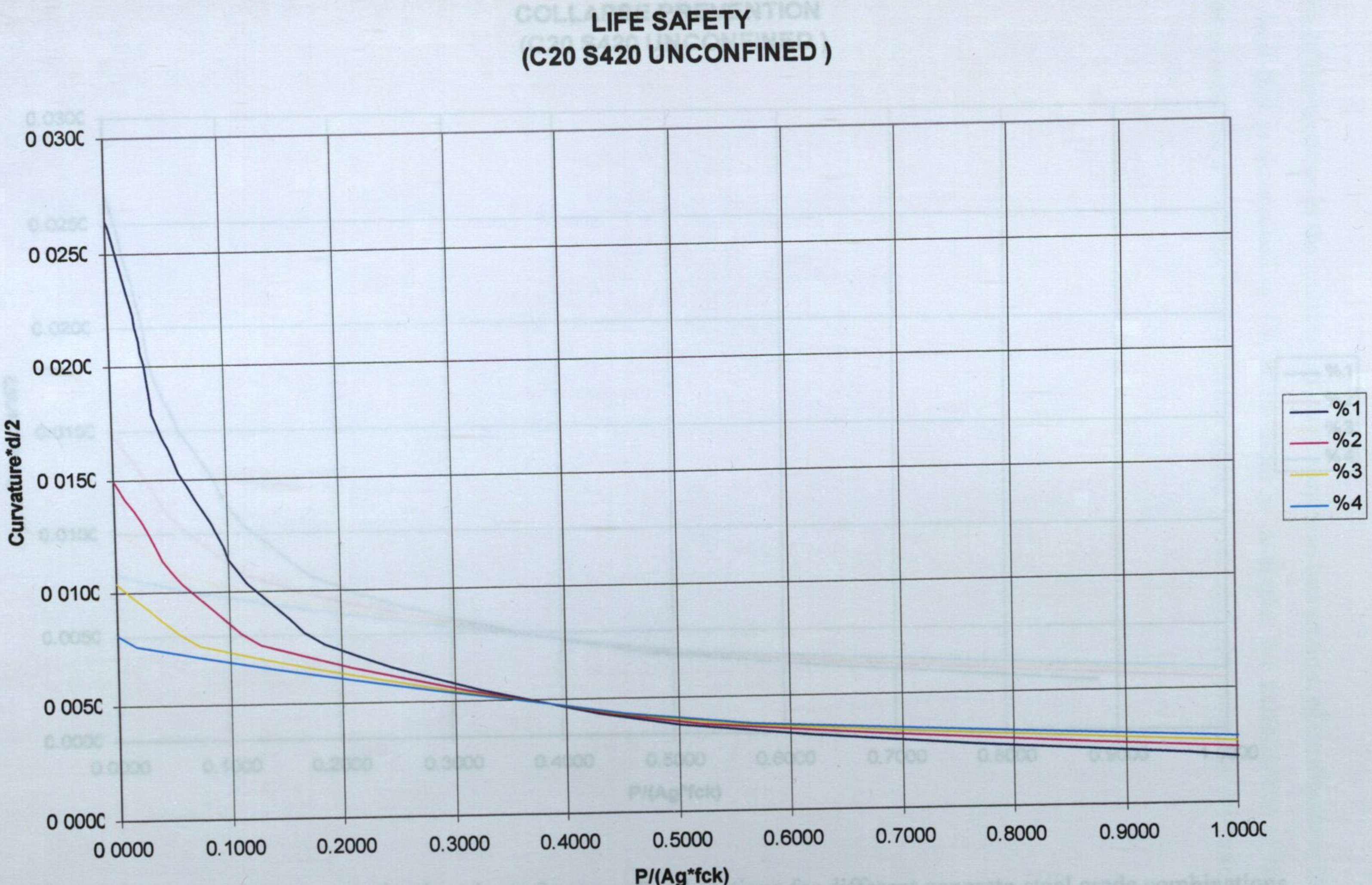
IMMEDIATE OCCUPANCY
(C20 S220 UNCONFINED)




LIFE SAFETY
(C20 S220 UNCONFINED)
0 0300
0 0250
0 0200
0 0150
0 0100
0 0050
0 0000
Curvature*d/2
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4


COLLAPSE PREVENTION
(C20 S220 UNCONFINED)
Curvature*d/2
0 0450
0 0400
0 0350
0 0300
0 0250
0 0200
0 0150
0 0100
0 0050
0 0000
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4


IMMEDIATE OCCUPANCY
(C20 S420 UNCONFINED)
Curvature*d/2
0 0090
0 0080
0 0070
0 0060
0 0050
0 0040
0 0030
0 0020
0 0010
0 0000
0 0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4


LIFE SAFETY
(C20 S420 UNCONFINED)
Curvature*d/2
0 0300
0 0250
0 0200
0 0150
0 0100
0 0050
0 0000
0 0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4

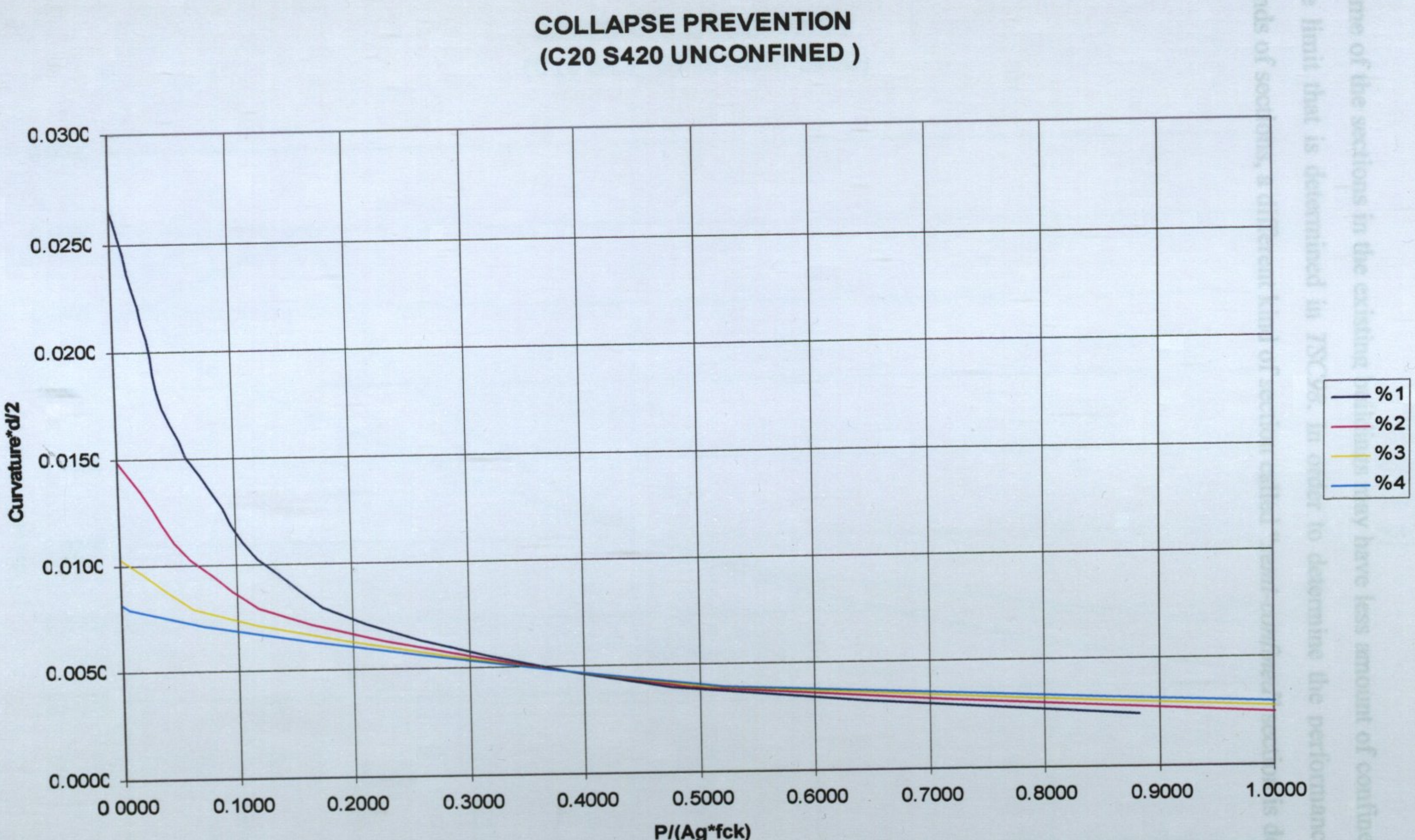
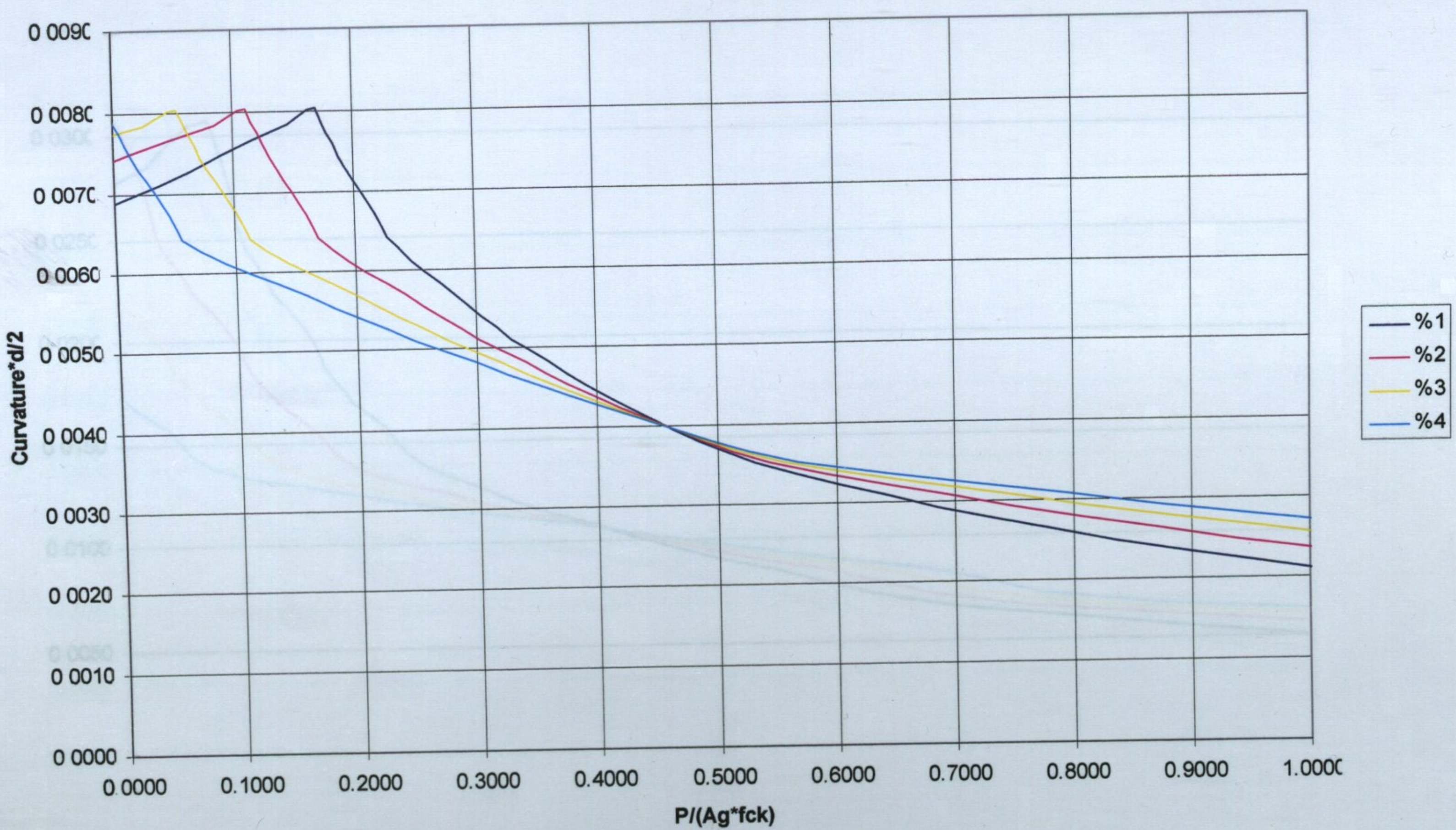


Figure 2.15. Several performance evaluation charts for unconfined sections for different concrete-steel grade combinations

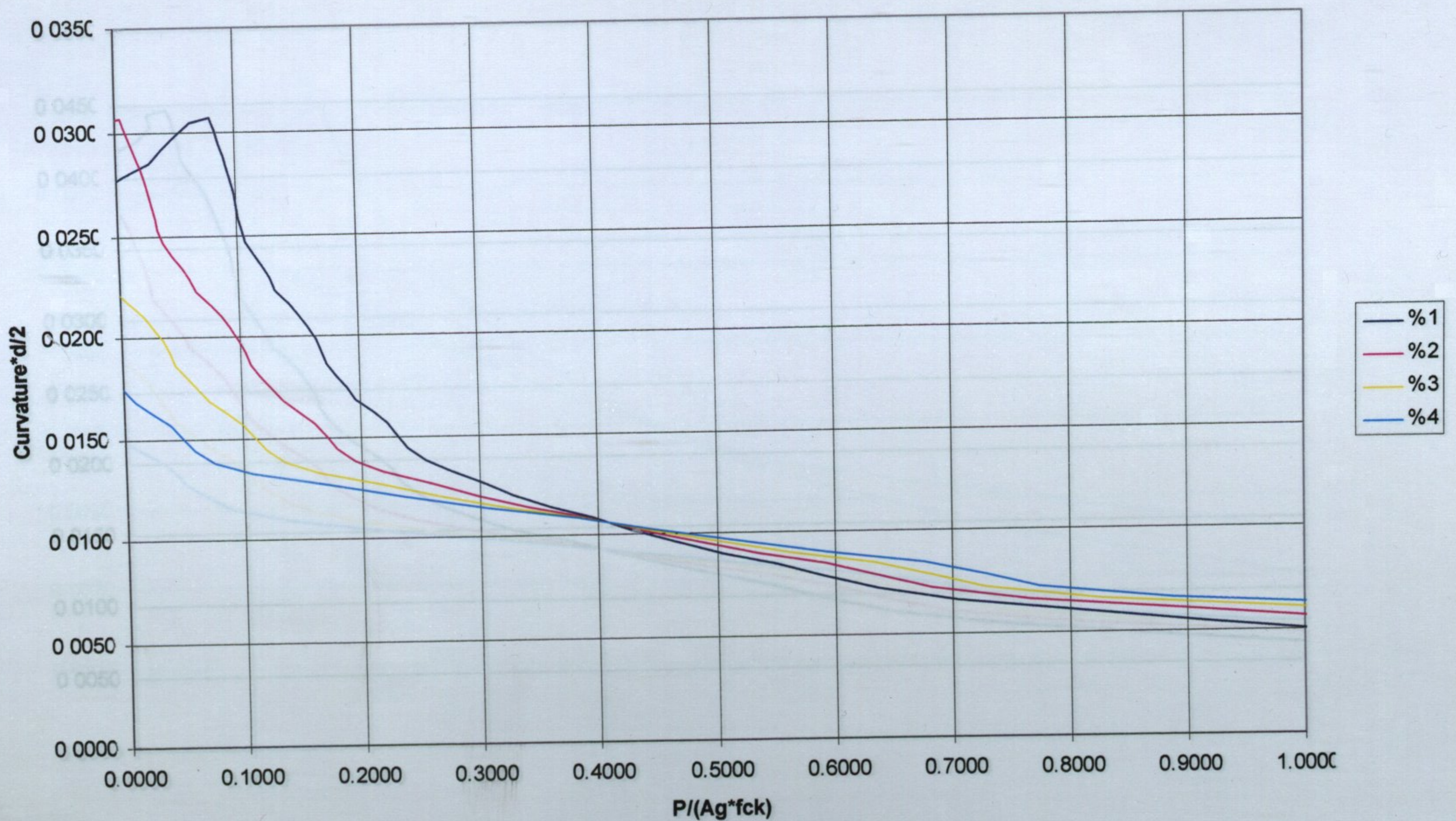
2.4.2. Semi Confined Sections

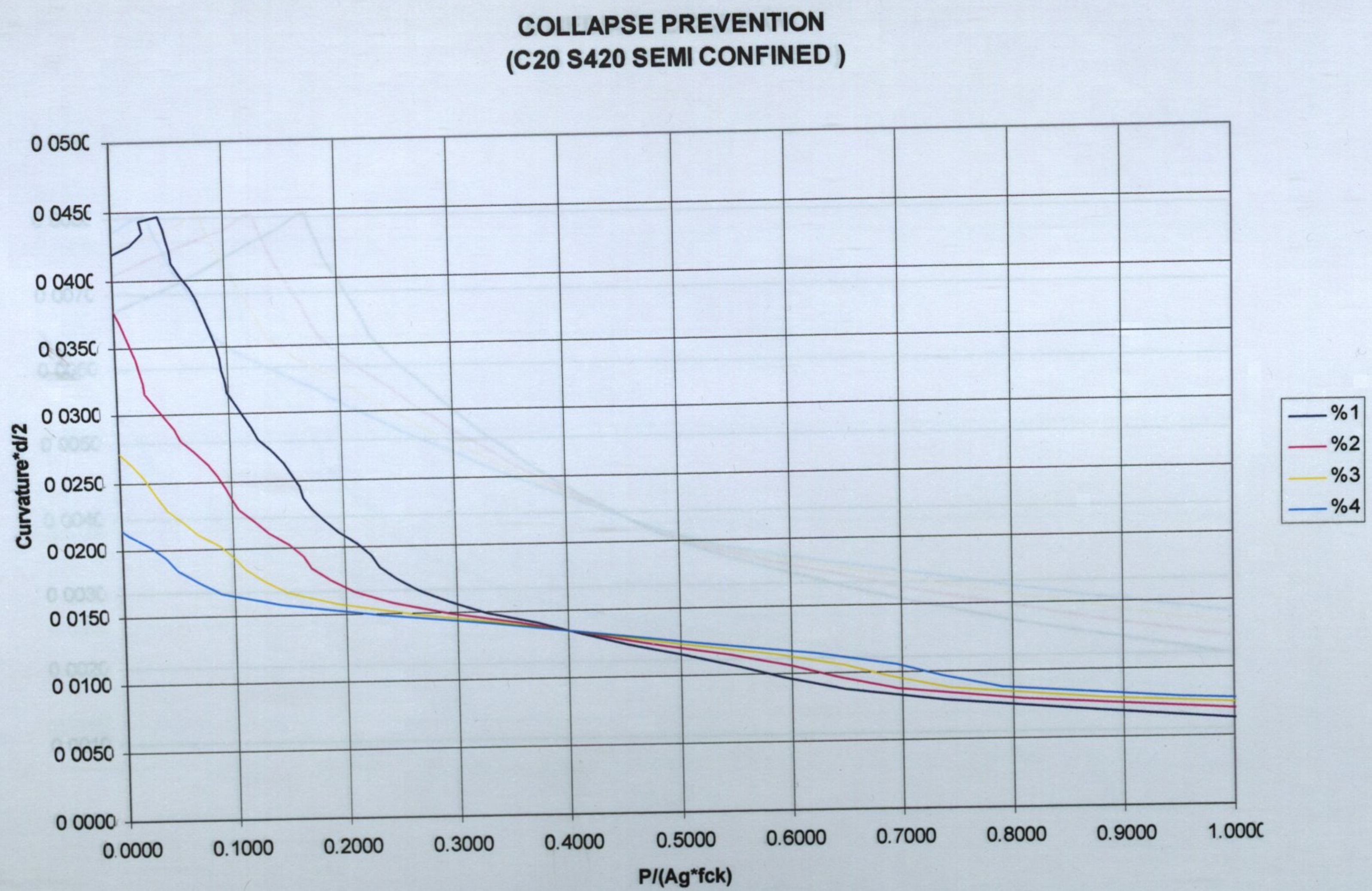
Some of the sections in the existing buildings may have less amount of confinement than the limit that is determined in *TSC98*. In order to determine the performance of these kinds of sections, a different kind of section called "*semi-confined*" section is determined.

IMMEDIATE OCCUPANCY
(C20 S420 SEMI CONFINED)

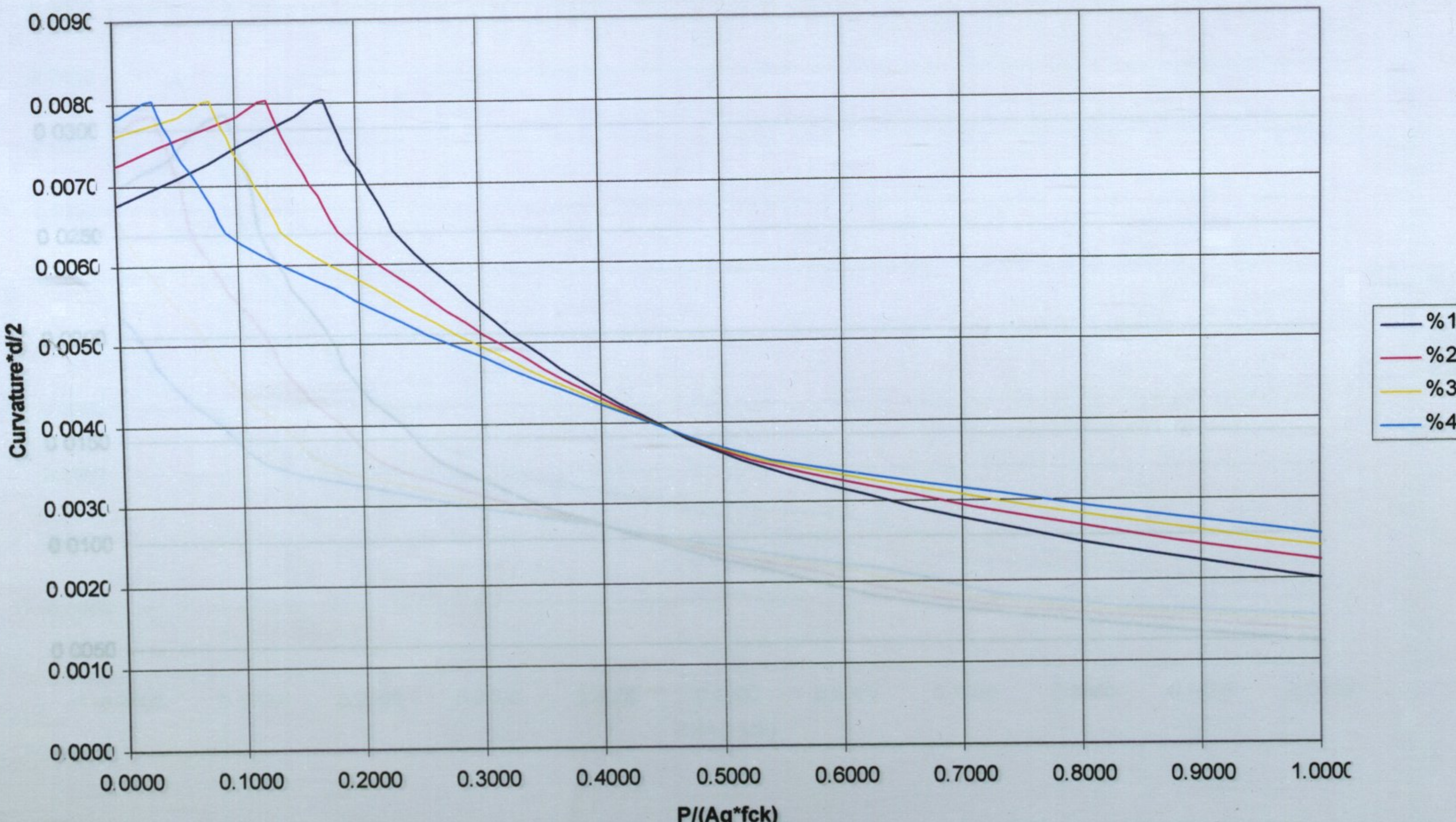


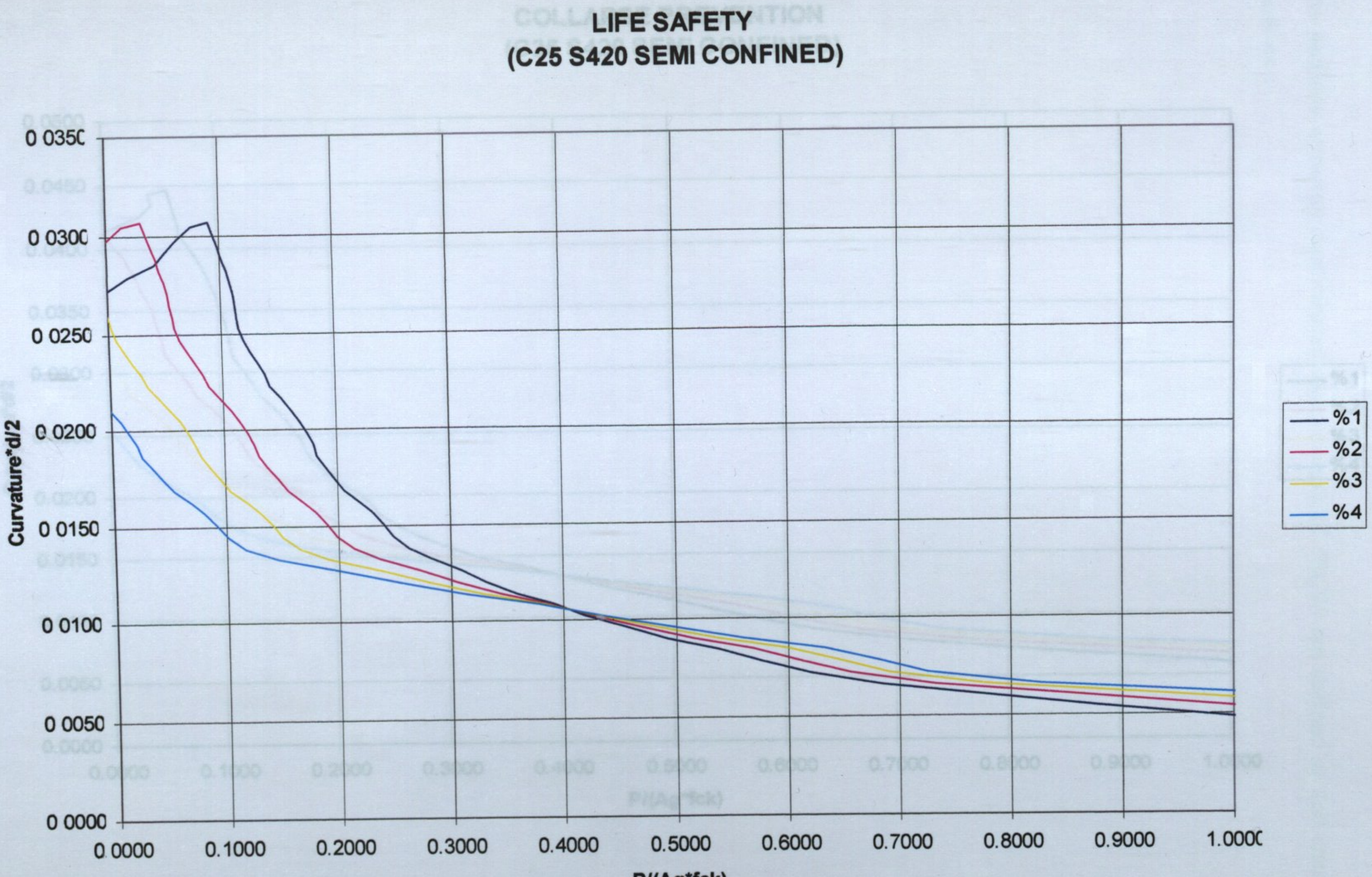
LIFE SAFETY
(C20 S420 SEMI CONFINED)




COLLAPSE PREVENTION
(C20 S420 SEMI CONFINED)
Curvature*d/2
0 0500
0 0450
0 0400
0 0350
0 0300
0 0250
0 0200
0 0150
0 0100
0 0050
0 0000
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4

IMMEDIATE OCCUPANCY
(C25 S420 SEMI CONFINED)




LIFE SAFETY
(C25 S420 SEMI CONFINED)
Curvature*d/2
0 0350
0 0300
0 0250
0 0200
0 0150
0 0100
0 0050
0 0000
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4

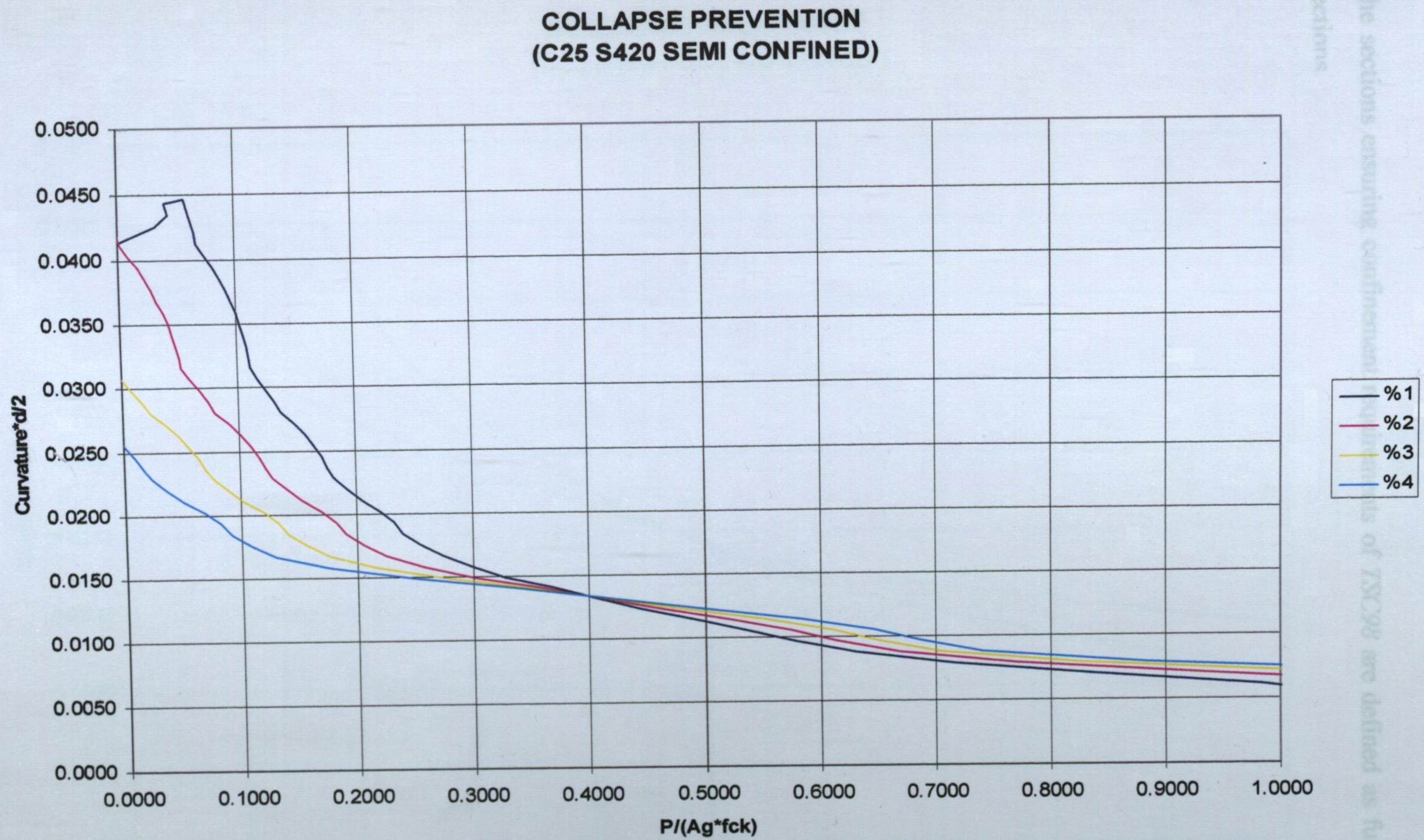
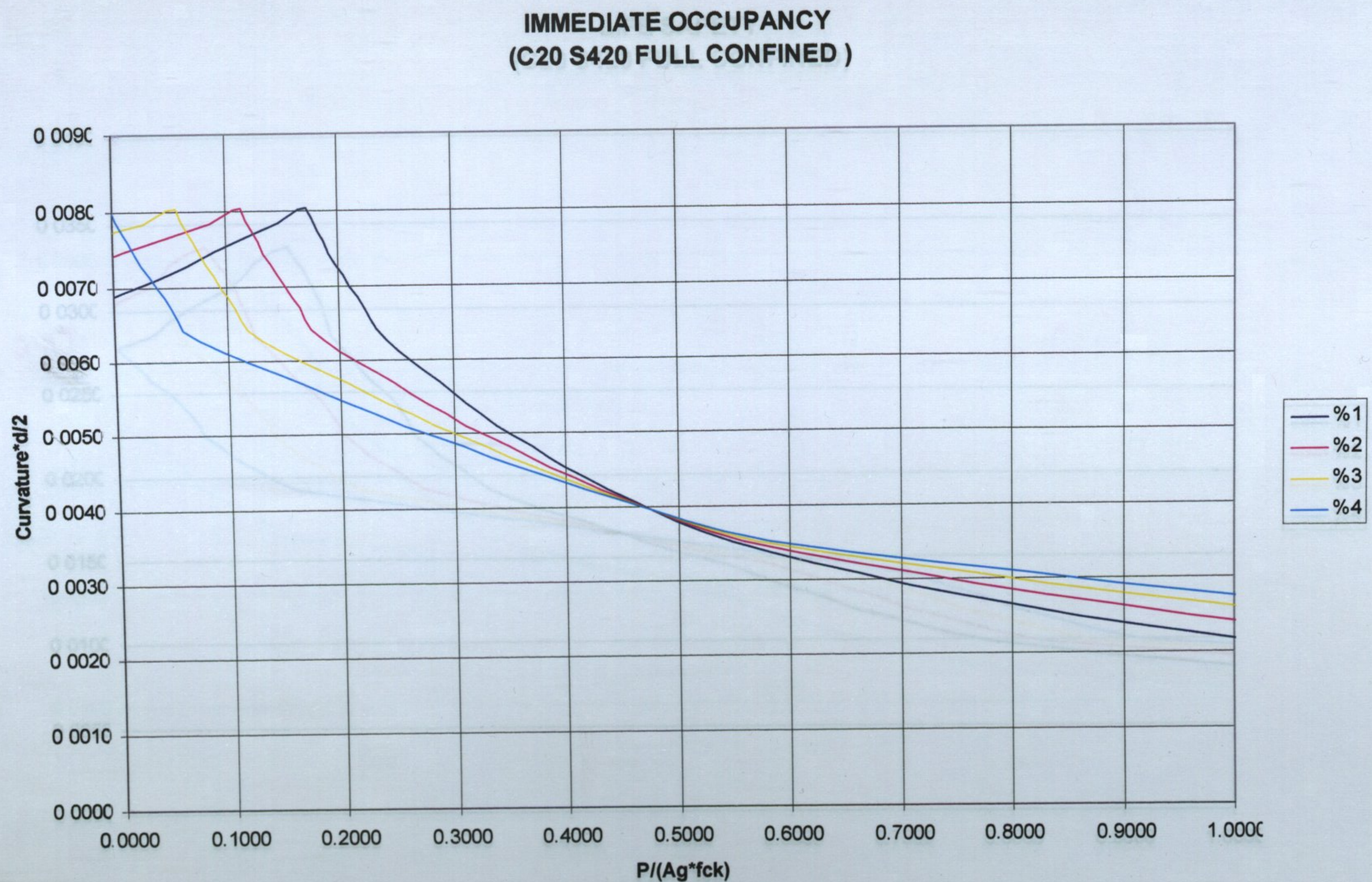


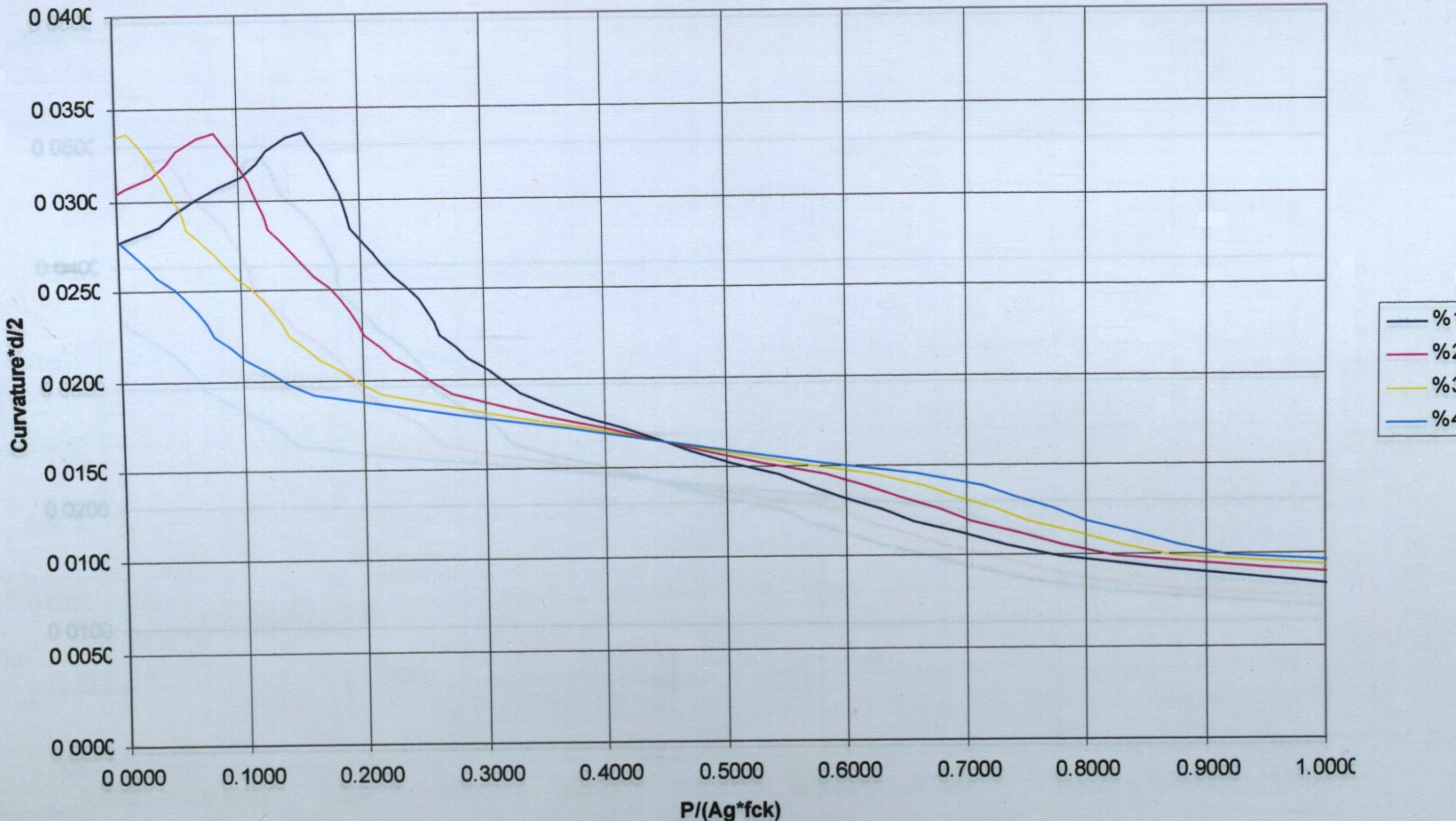
Figure 2.16. Several performance evaluation charts for semi confined sections for different concrete-steel grade combinations

2.4.3. Full Confined (Confined) Sections

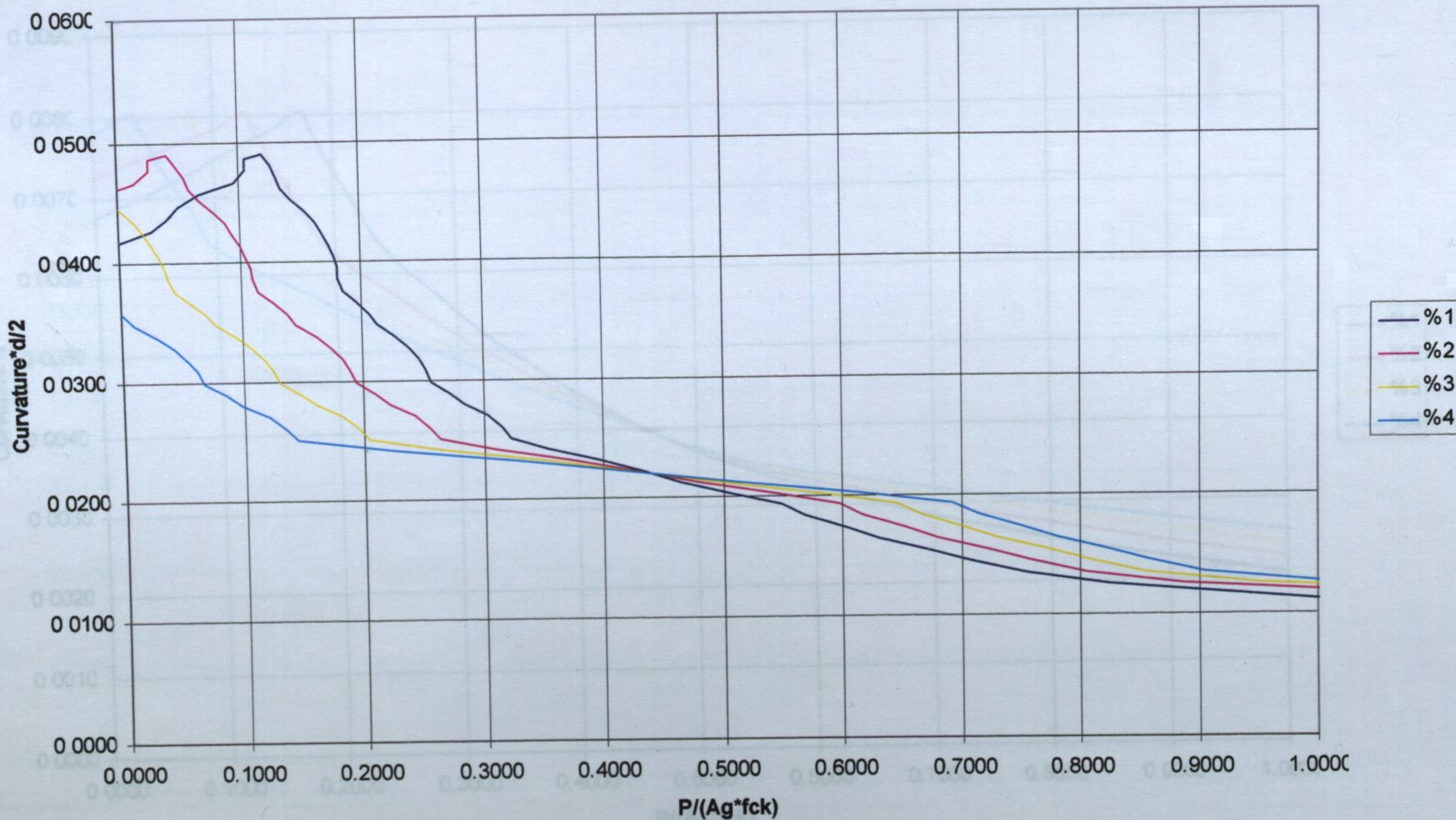
The sections ensuring confinement requirements of *TSC98* are defined as full confined sections.

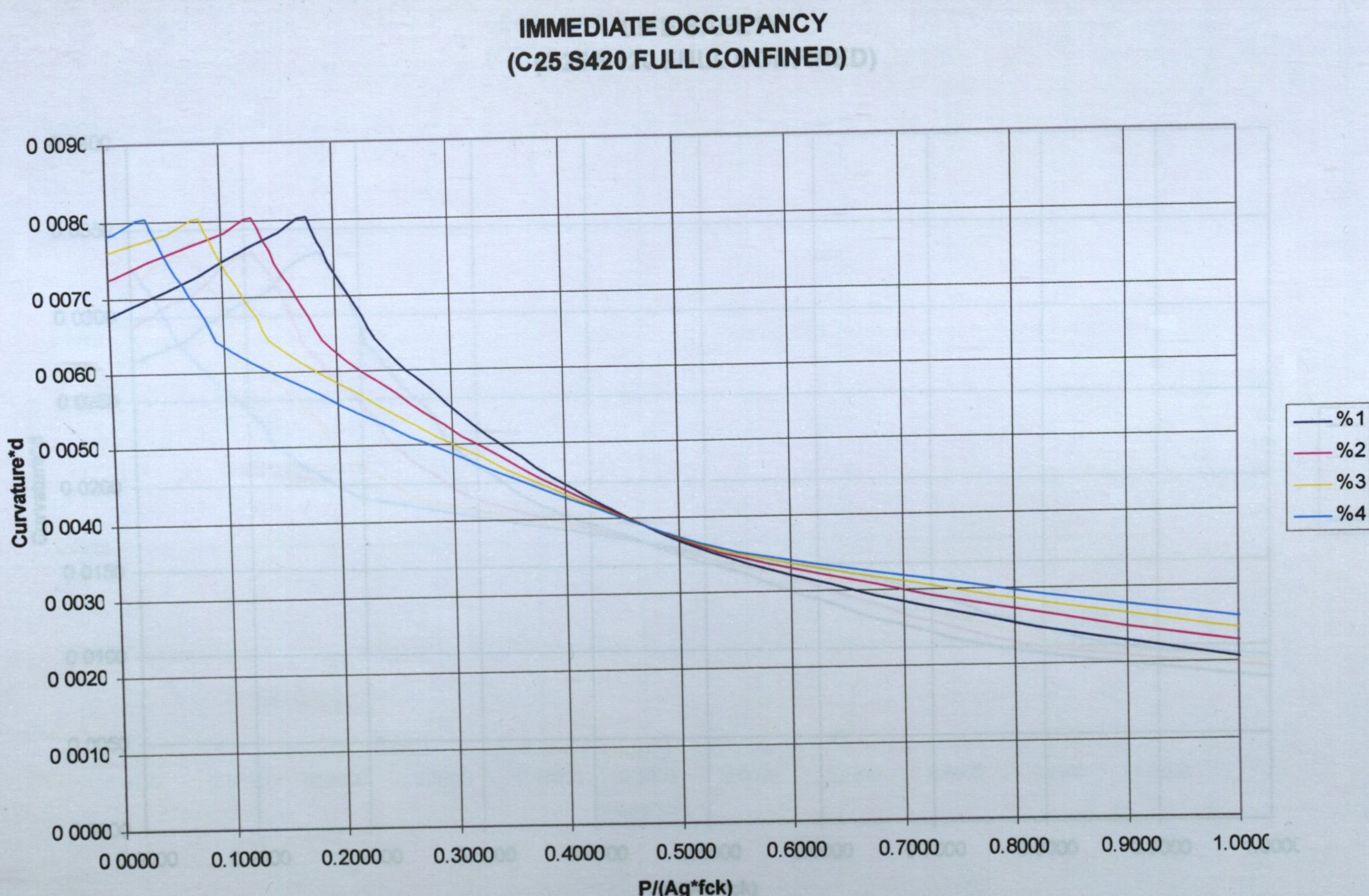

IMMEDIATE OCCUPANCY
(C20 S420 FULL CONFINED)
Curvature*d/2
0 0090
0 0080
0 0070
0 0060
0 0050
0 0040
0 0030
0 0020
0 0010
0 0000
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
P/(Ag*fck)
%1
%2
%3
%4

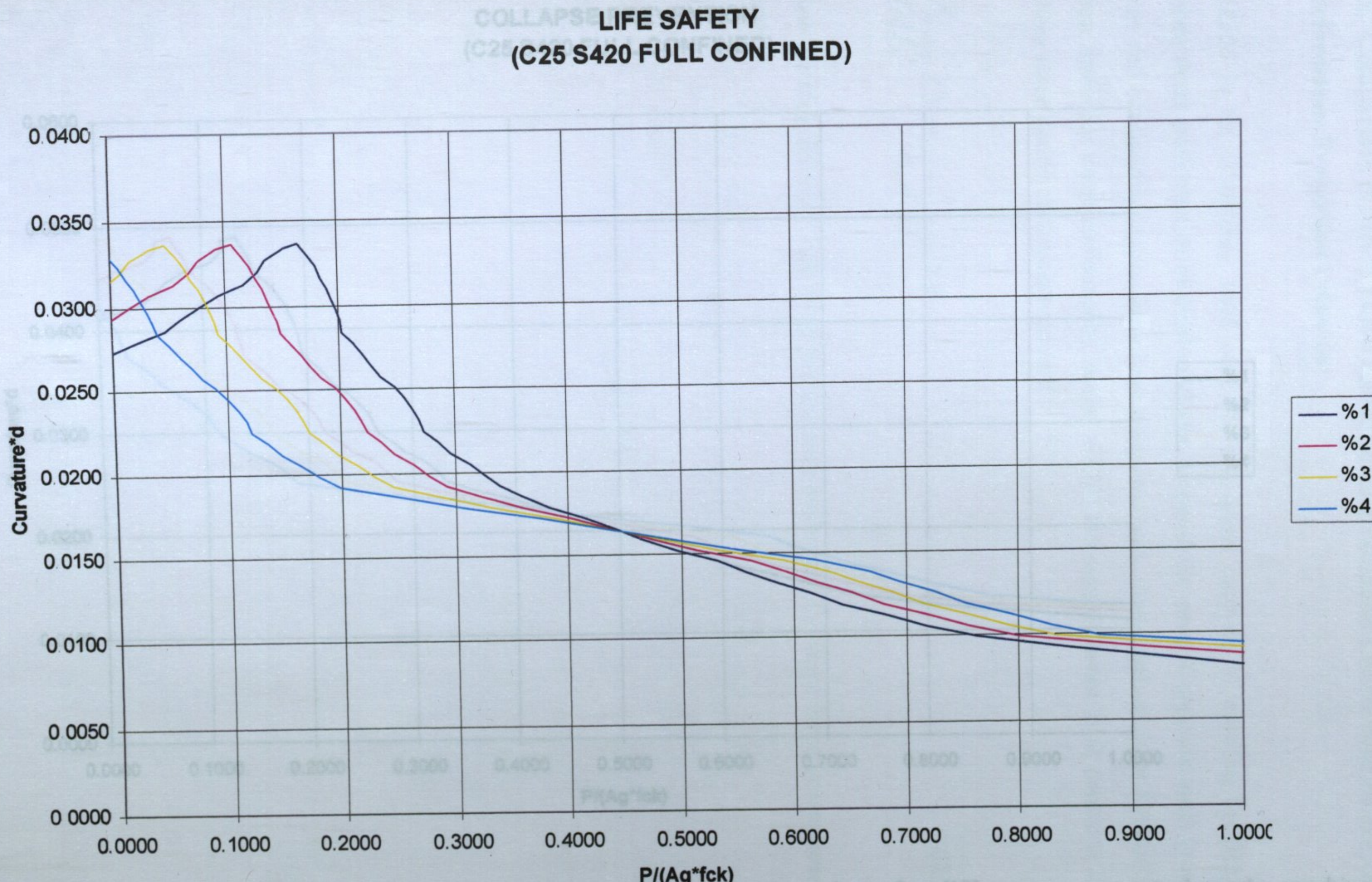
LIFE SAFETY
(C20 S420 FULL CONFINED)



COLLAPSE PREVENTION
(C20 S420 FULL CONFINED)




IMMEDIATE OCCUPANCY
(C25 S420 FULL CONFINED)
Curvature*d
0 0090
0 0080
0 0070
0 0060
0 0050
0 0040
0 0030
0 0020
0 0010
0 0000
P/(Ag*fck)
0 0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000
%1
%2
%3
%4


LIFE SAFETY
(C25 S420 FULL CONFINED)
Curvature*d
P/(Ag*fck)
%1
%2
%3
%4
0.0000
0.0050
0.0100
0.0150
0.0200
0.0250
0.0300
0.0350
0.0400
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
0.8000
0.9000
1.0000

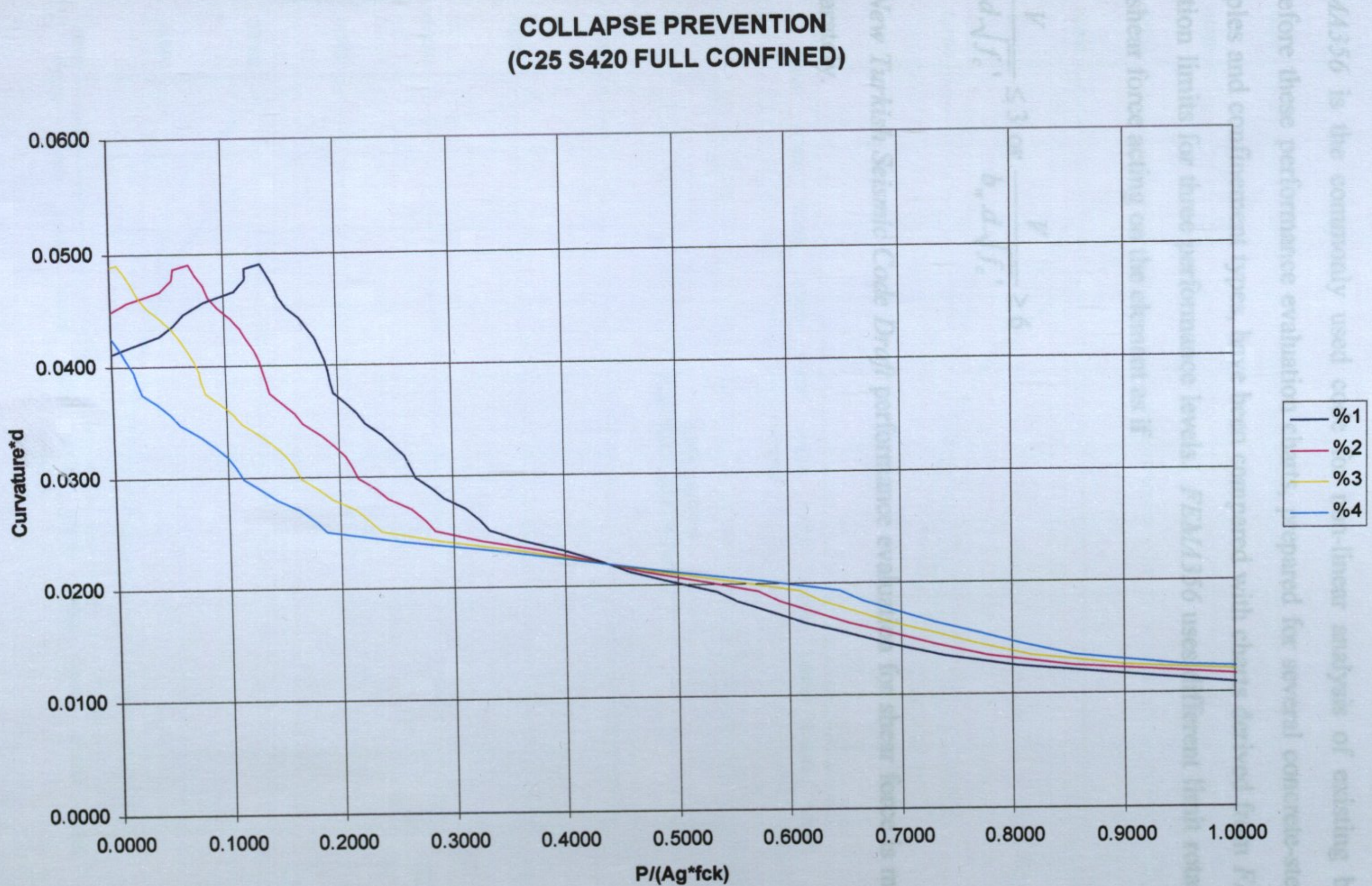


Figure 2.17. Several performance evaluation charts for full confined sections for different concrete-steel grade combinations

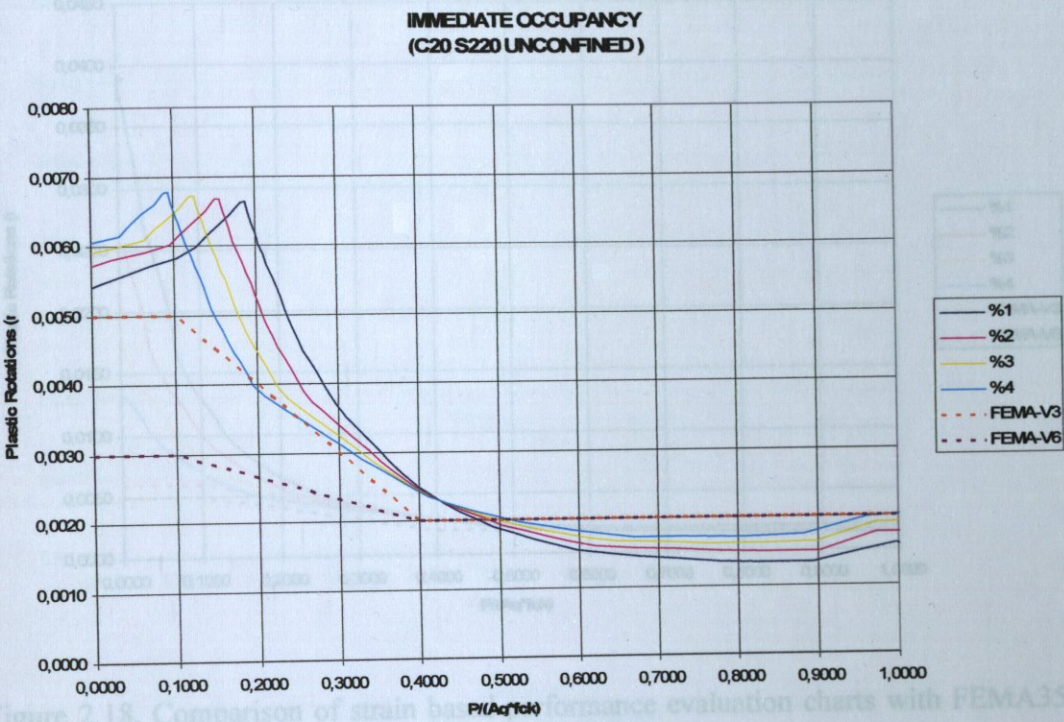
2.5. Comparison of Strain Based Performance Evaluation Charts with *FEMA356* Performance Evaluation Criterion

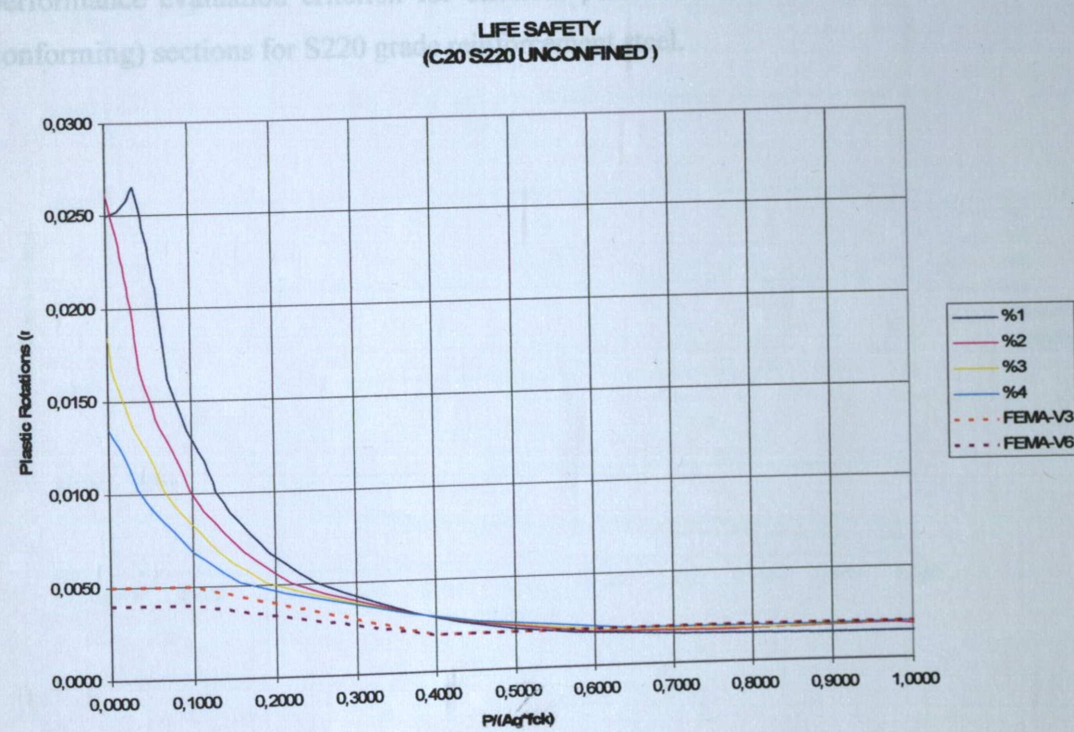
FEMA356 is the commonly used code for non-linear analysis of existing buildings therefore these performance evaluation charts, prepared for several concrete-steel grade couples and confinement types, have been compared with charts derived from *FEMA356* rotation limits for three performance levels. *FEMA356* uses different limit rotations for the shear force acting on the element as if

$$\frac{V}{b_w.d.\sqrt{f_c'}} \leq 3 \text{ or } \frac{V}{b_w.d.\sqrt{f_c'}} \geq 6 \tag{2.3}$$

In *New Turkish Seismic Code Draft* performance evaluation for shear force is mentioned separately.

2.5.1. Unconfined (Non-conforming Sections)





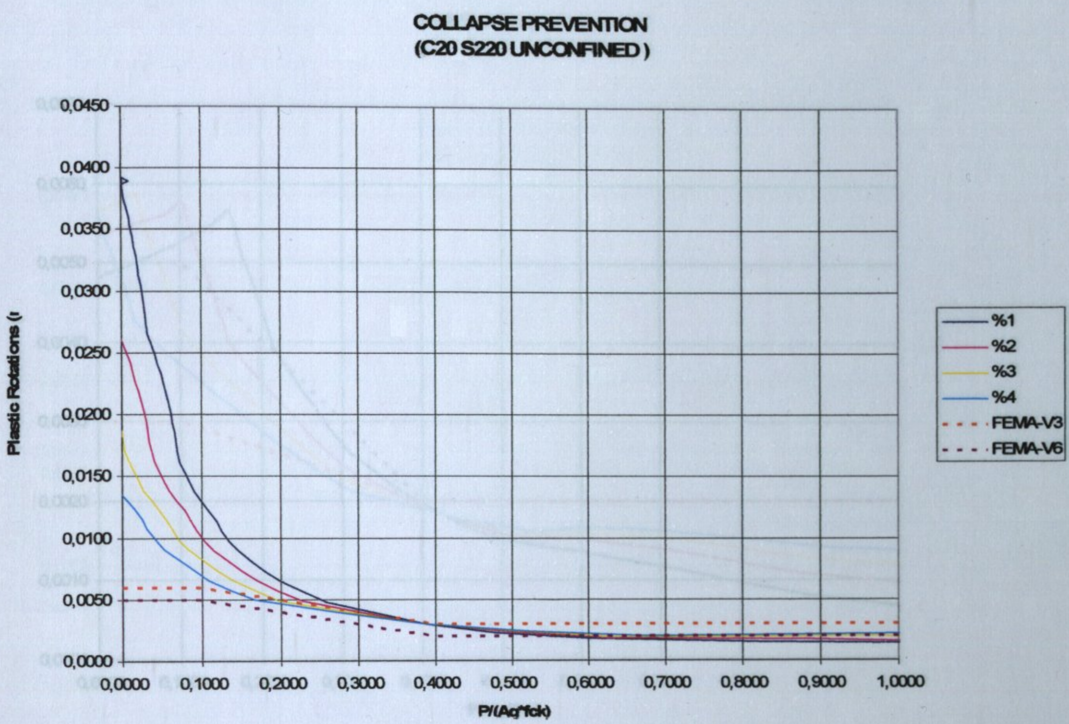
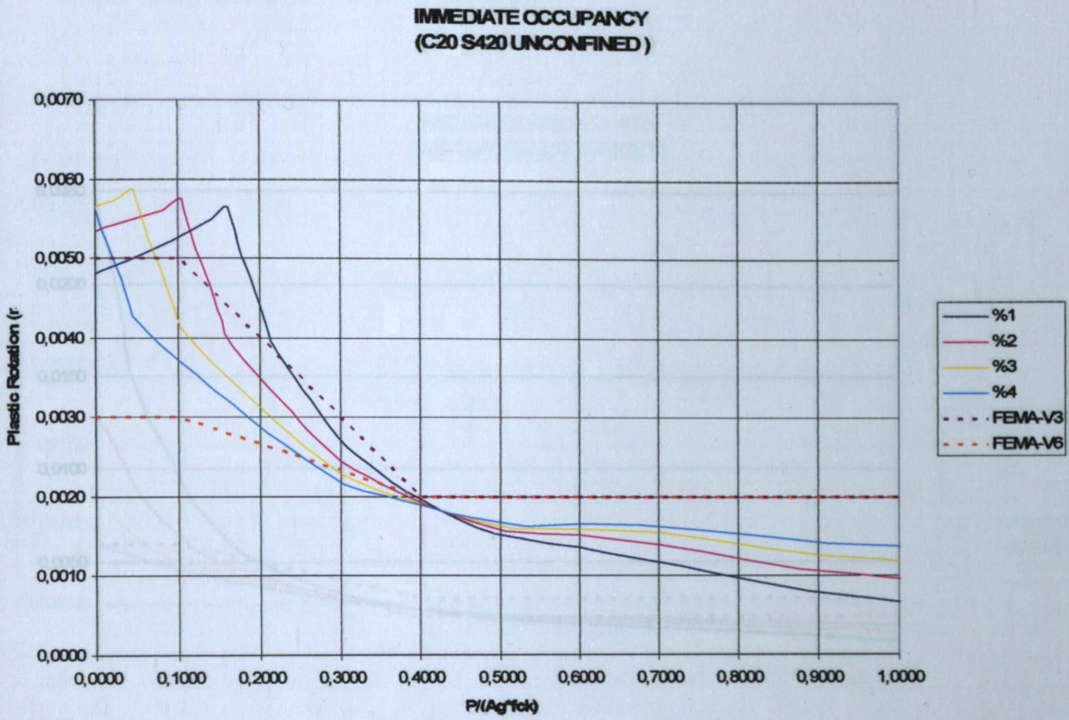
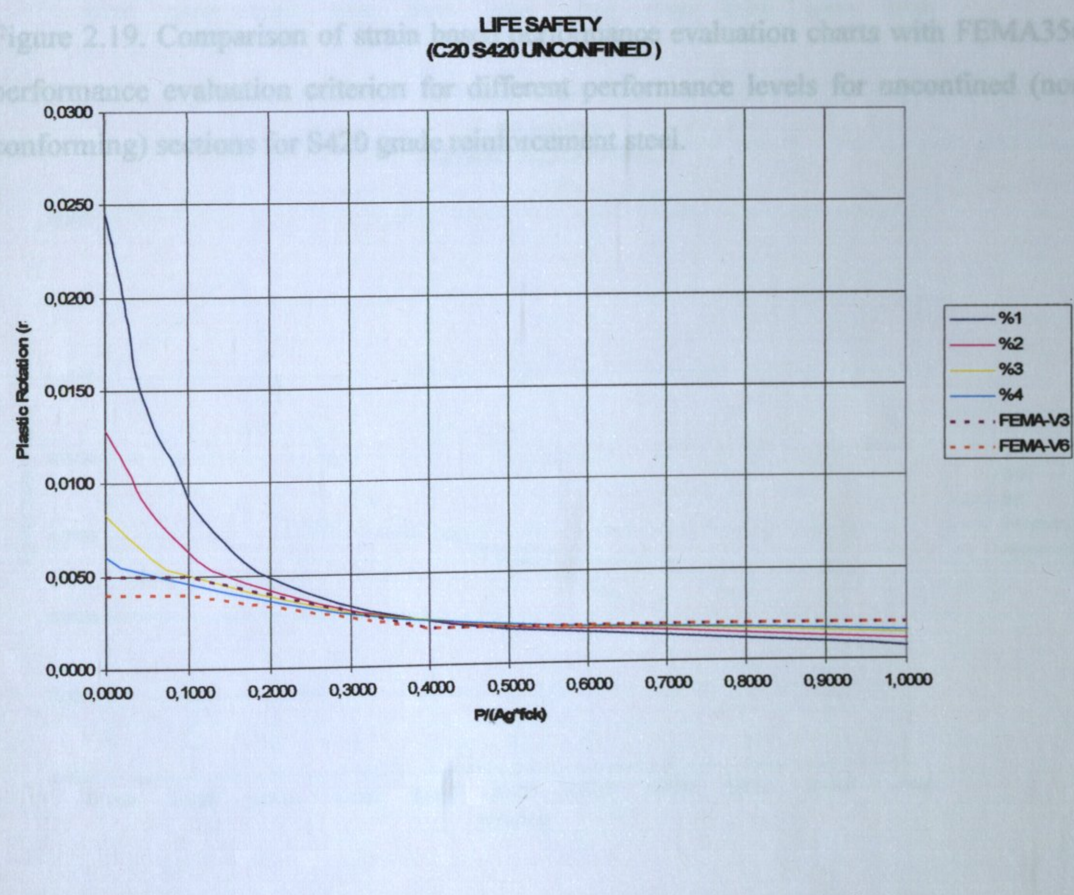


Figure 2.18. Comparison of strain based performance evaluation charts with FEMA356 performance evaluation criterion for different performance levels for unconfined (non conforming) sections for S220 grade reinforcement steel.


IMMEDIATE OCCUPANCY
(C20 S420 UNCONFINED)
0,0070
0,0060
0,0050
0,0040
0,0030
0,0020
0,0010
0,0000
Plastic Rotation (r
0,0000
0,1000
0,2000
0,3000
0,4000
0,5000
0,6000
0,7000
0,8000
0,9000
1,0000
P/(Ag*fck)
%1
%2
%3
%4
FEMA-V3
FEMA-V6


LIFE SAFETY
(C20 S420 UNCONFINED)
0,0300
0,0250
0,0200
0,0150
0,0100
0,0050
0,0000
Plastic Rotation (r
0,0000
0,1000
0,2000
0,3000
0,4000
0,5000
0,6000
0,7000
0,8000
0,9000
1,0000
P/(Ag*fck)
%1
%2
%3
%4
FEMA-V3
FEMA-V6

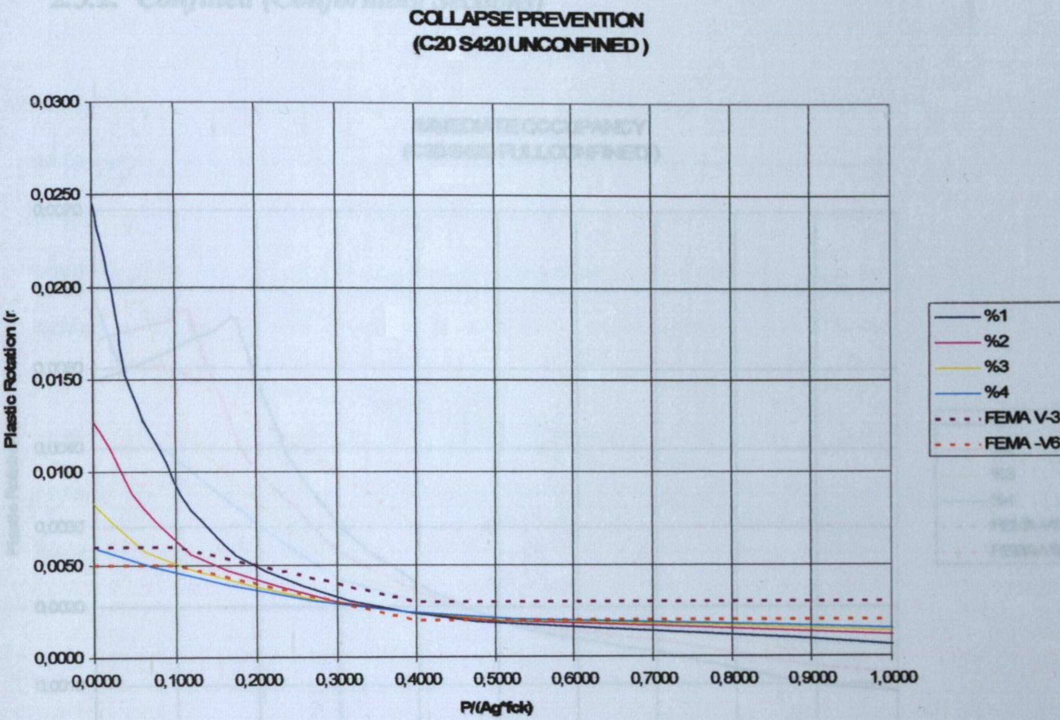
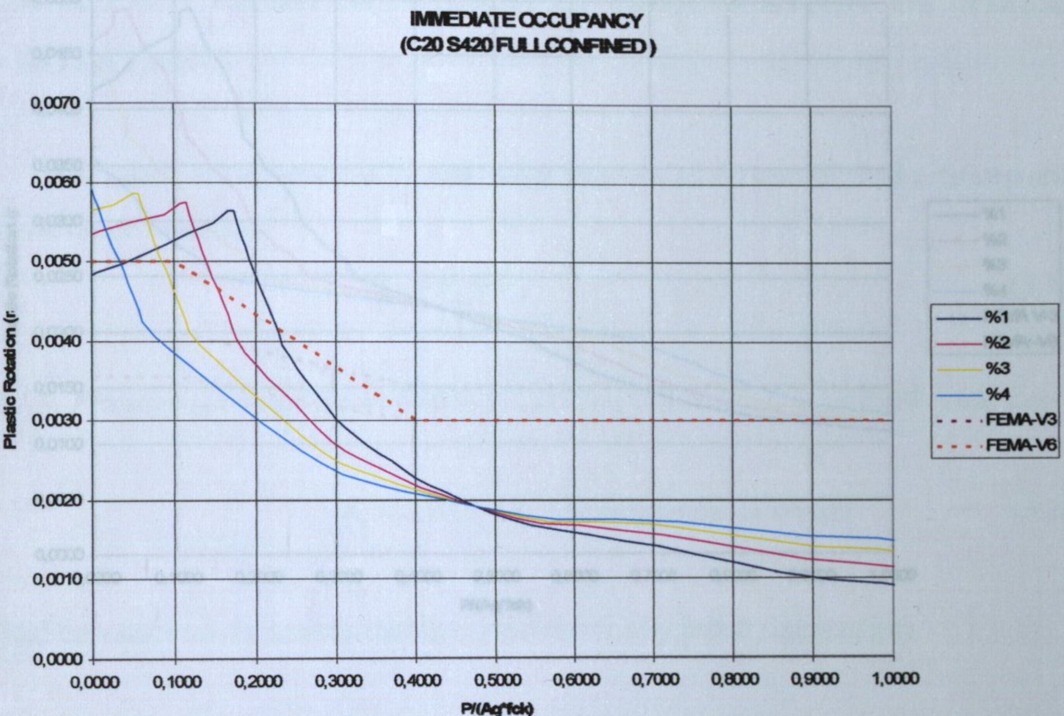
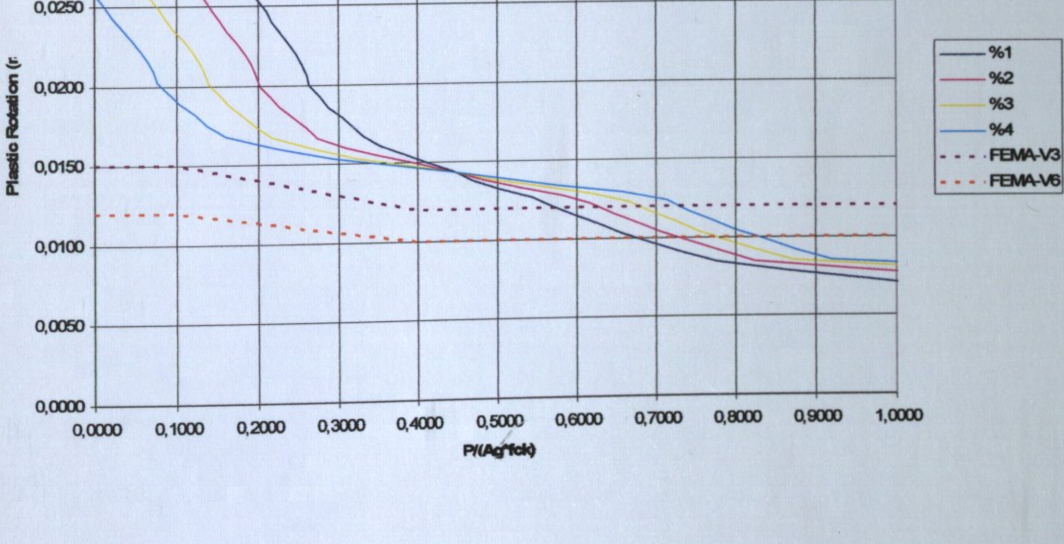


Figure 2.19. Comparison of strain based performance evaluation charts with FEMA356 performance evaluation criterion for different performance levels for unconfined (non conforming) sections for S420 grade reinforcement steel.

2.5.2. *Confined (Conforming Sections)*





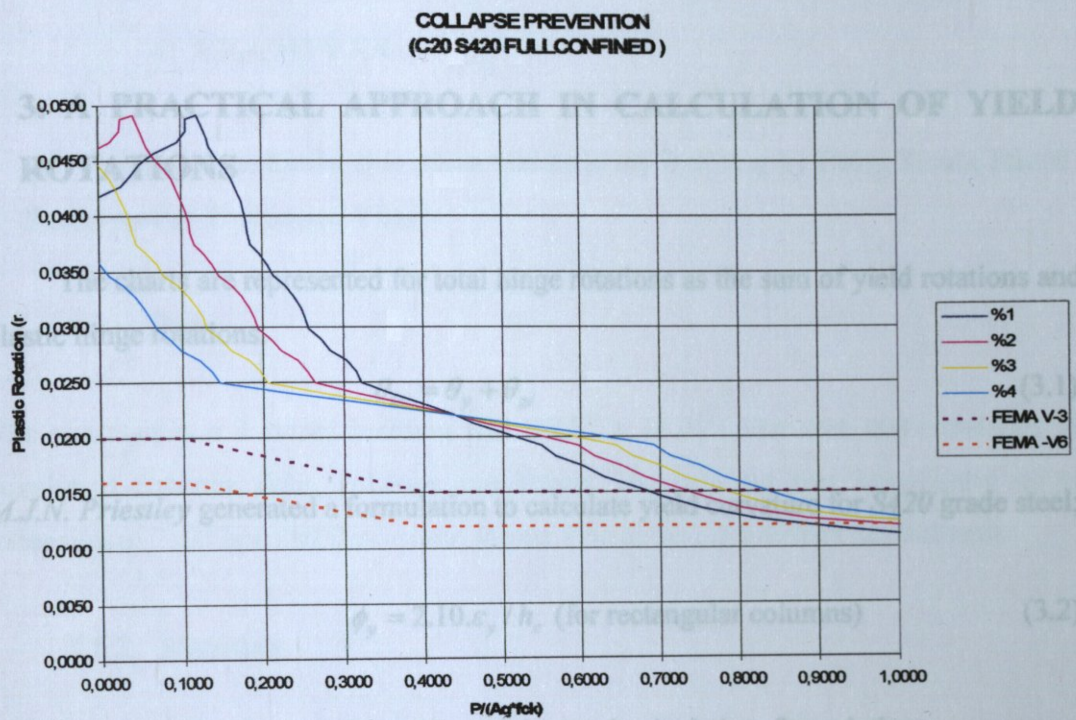


Figure 2.20. Comparison of strain based performance evaluation charts with FEMA356 performance evaluation criterion for different performance levels for confined (conforming) sections for S420 grade reinforcement steel.

3. A PRACTICAL APPROACH IN CALCULATION OF YIELD ROTATIONS

The charts are represented for total hinge rotations as the sum of yield rotations and plastic hinge rotations.

$$\theta_{tot} = \theta_y + \theta_{pl} \tag{3.1}$$

M.J.N. Priestley generated a formulation to calculate yield curvature for *S420* grade steel;

$$\phi_y = 2.10.\varepsilon_y / h_c \text{ (for rectangular columns)} \tag{3.2}$$

Yield curvature can be transformed into yield rotation by below formulation;

$$\theta_y = \phi_y.L_p \tag{3.3}$$

4. EXAMPLES

4.1. Performance Evaluation of An Old Existing Building by Using Strain Based Performance Evaluation Charts

4.1.1. Data

The structure is a 4 storey building with $42{,}5^{m}$ x 18.00^{m} plan area and consisting of reinforced concrete slabs, columns and beams. The building was investigated as an extension of 2003 Istanbul Earthquake Master Plan at the pilot district Zeytinburnu.

4.1.2. Materials

<u>*Concrete*</u>

$f_{ck} = 14$ MPa

$E_c = 3250\sqrt{(fck)} + 14000$ MPa

$E_c = 3250\sqrt{14} + 14000 = 26160\text{MPa} = 2.62\text{ E7 kN/m2}$

<u>*Steel*</u>

Longitudinal Steel: *S420* $f_{yk} = 420$ MPa

Transverse Steel : *S420* $f_{yk} = 420$ MPa

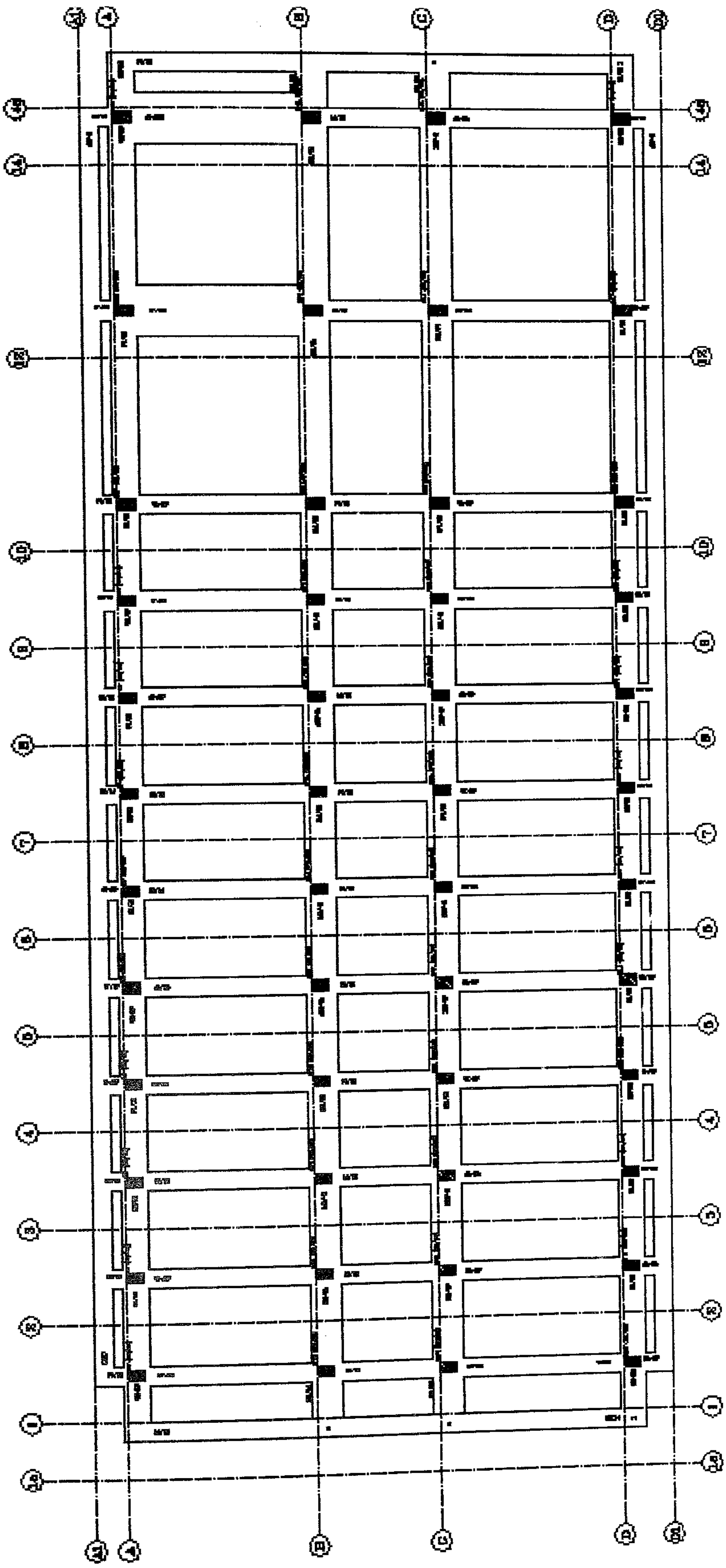


Figure 4.1. Typical floor plan

4.1.3. Effecting Vertical Loads

Reinforced Concrete Slab	0.10*25.00	=2.50 kN/m^2
Superimposed Dead	0.06*20.00	= 1.20 kN/m^2
Interior Separating Wall		= 1.50 kN/m^2
External Wall		= 3.80 kN/m^2
Live		= 2.00 kN/m^2

Live Load participation factor for earthquake loading has been taken 0.30

4.1.4. Elastic Response Spectrum

Elastic Response Spectrum given in *TSC98* has been considered at *1st Seismic Zone* with building importance factor *I=1.0* and T_a, T_b values given below

T_A= 0.15 sec.

T_B=0.60 sec.

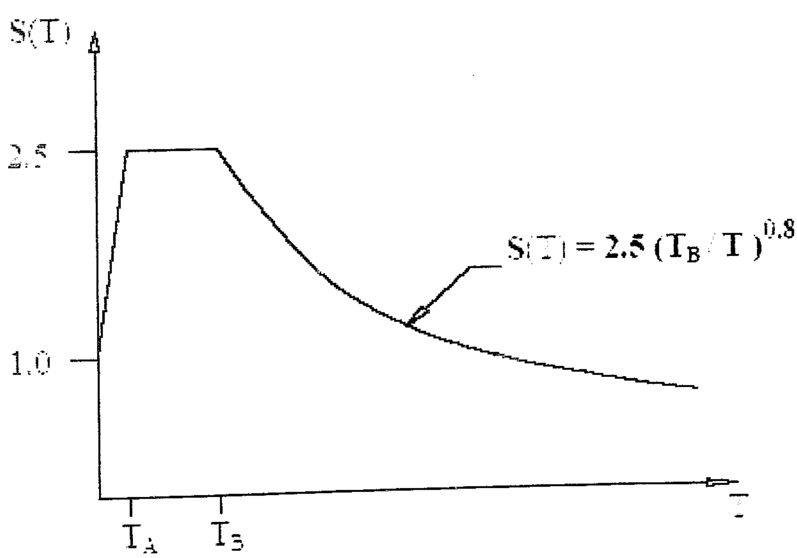


Figure 4.2. Elastic Response Spectrum (TSC98)

4.1.5. Non-Linear Static Pushover Analysis of the Structure

The structure has been modeled using a nonlinear structural analysis software '*SAP2000*'. Plastic hinge properties have been defined by considering column '*axial force – bending moment*' and beam '*bending moment – curvature*' relations and then these properties have been assigned to the related beam and column end points. Mode invariant pushover analysis has been performed for both directions.

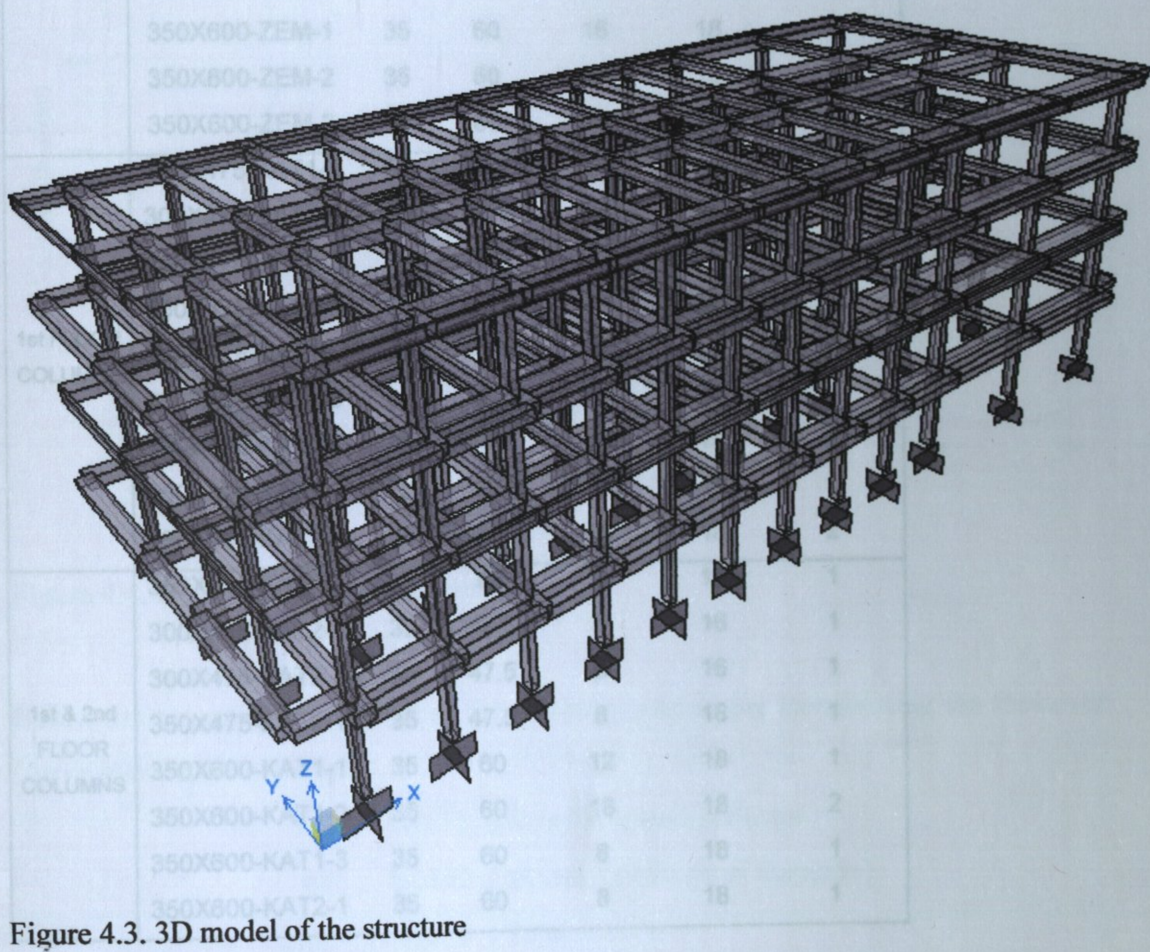


Figure 4.3. 3D model of the structure

Table 4.1. Column details

Storey	SectionName	Dimensions		NumBars	BarDia	ReinfRatio
Text	Text	cm	cm	Unitless	mm	%
GROUND FLOOR COLUMNS	300X475-ZEM-1	30	47.5	12	18	2
	300X475-ZEM-2	30	47.5	14	18	3
	300X475-ZEM-3	30	47.5	8	18	1
	300X475-ZEM-4	30	47.5	8	16	1
	350X475-ZEM-1	35	47.5	12	18	2
	350X475-ZEM-2	35	47.5	18	18	3
	350X600-ZEM-1	35	60	16	18	2
	350X600-ZEM-2	35	60	18	18	2
	350X600-ZEM-3	35	60	18	18	2
1st FLOOR COLUMNS	300X475-KAT1-1	30	47.5	10	18	2
	300X475-KAT1-2	30	47.5	12	18	2
	300X475-KAT1-3	30	47.5	6	16	1
	350X475-KAT1-1	35	47.5	10	18	2
	350X475-KAT1-2	35	47.5	18	18	3
	350X475-KAT1-3	35	47.5	8	18	1
	350X475-KAT1-4	35	47.5	10	18	2
	350X600-KAT1-1	35	60	12	18	1
	350X600-KAT1-2	35	60	18	18	2
1st & 2nd FLOOR COLUMNS	350X600-KAT1-3	35	60	8	18	1
	300X475-KAT2-1	30	47.5	8	16	1
	300X475-KAT2-2	30	47.5	6	16	1
	350X475-KAT2-1	35	47.5	8	18	1
	350X600-KAT1-1	35	60	12	18	1
	350X600-KAT1-2	35	60	18	18	2
	350X600-KAT1-3	35	60	8	18	1
	350X600-KAT2-1	35	60	8	18	1

<u>*Pushover Analysis Results for the Direction X*</u>

Pushover Curve and Capacity Diagram obtained after nonlinear analysis is given below.

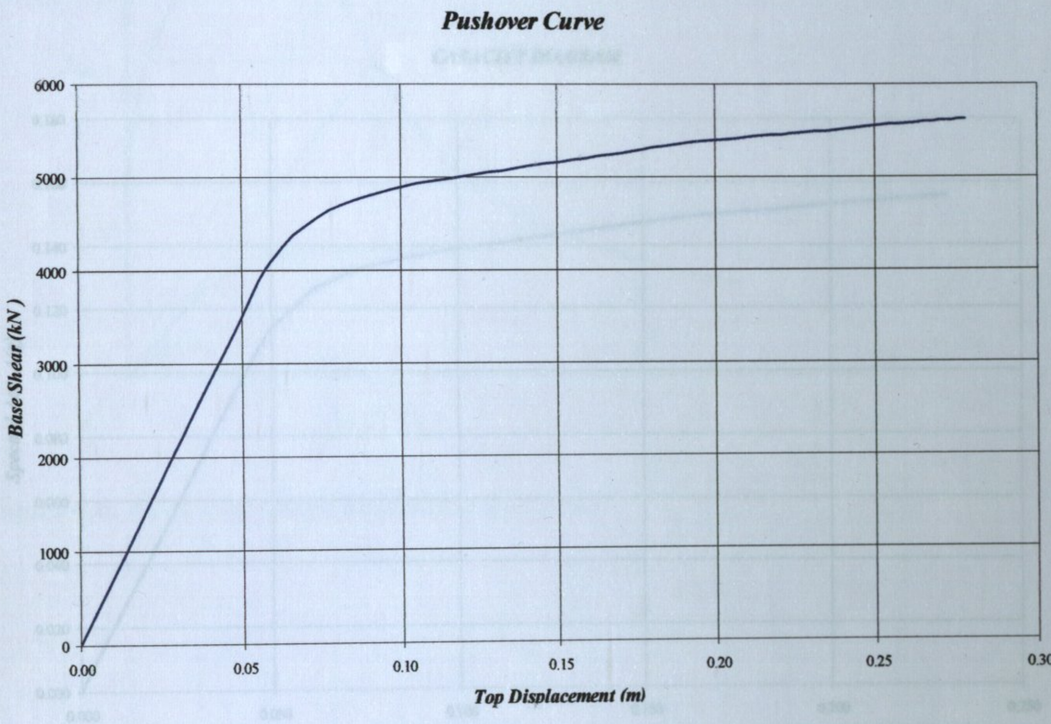


Figure 4.4. X directional pushover curve

Below are the modal parameters of the structure used during transforming the Pushover Curve into Capacity Diagram.

$\Gamma_1=$ 60.675 (Mass Participation Factor)

$\phi_r=$ 0.0199 (Modal Amplitude of the node)

$\Sigma M=$ 4060 t (Total Mass)

M(%)= 90.68 (Participating Mass Ratio)

$M_{eff}=$ 3682 t (Effective Mass)

And the circular frequency and period of the system using the cracked stiffness properties with the modal parameters given above are as below.

ω^2= 23.05 rad/sec

T= 1.309 sec

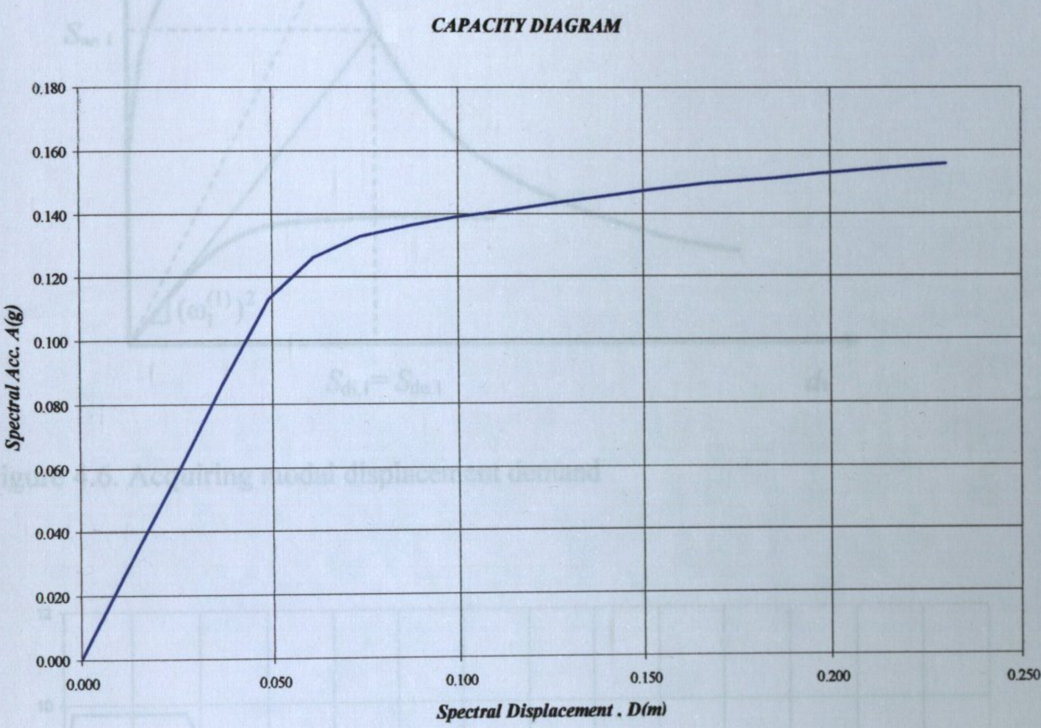


Figure 4.5. X directional capacity diagram

The procedures to attain the modal performance displacement value is defined at *New Turkish Seismic Code Draft section 13.6.8*

According to NTSCD, as the period of the linearly elastic system T_1, is greater than the T_B value defined in S*ection 4.1.4.* ($T_1 \geq T_B$), nonlinear spectral displacement $S_{di,1}$ is equal to the linear elastic spectral displacement $S_{de,1}$ of the equivalent linearly elastic system according to the equal displacement rule. Below charts are modal capacity diagram and spectral displacement – spectral acceleration charts drawn together in *New Turkish Seismic Code Draft* and for this example.

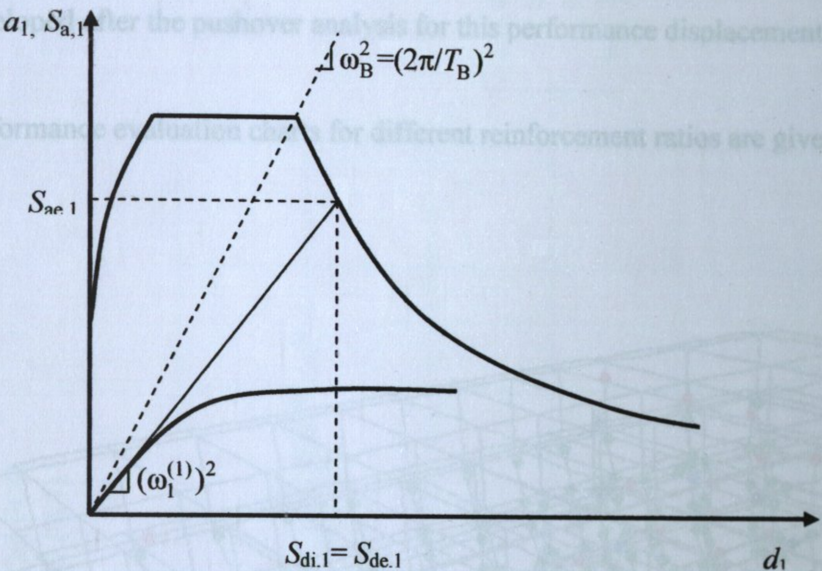


Figure 4.6. Acquiring modal displacement demand

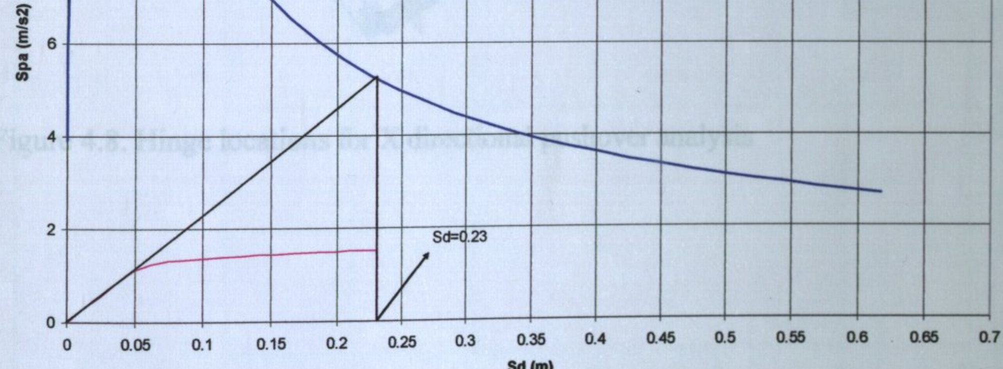


Figure 4.7. Acquiring performance displacement for X direction

The performance displacement (S_d) of the structure has been found to be 0.23^m. And performance evaluation for the structure is done by considering the plastic hinge rotations developed after the pushover analysis for this performance displacement.

Performance evaluation charts for different reinforcement ratios are given below.

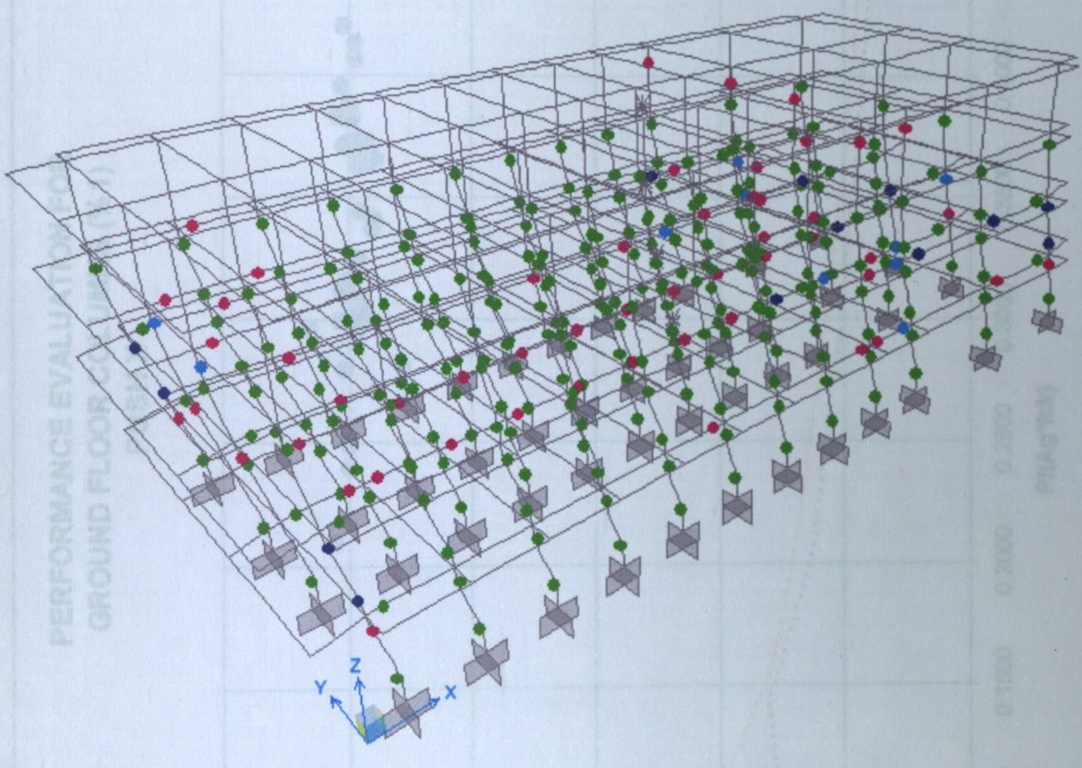
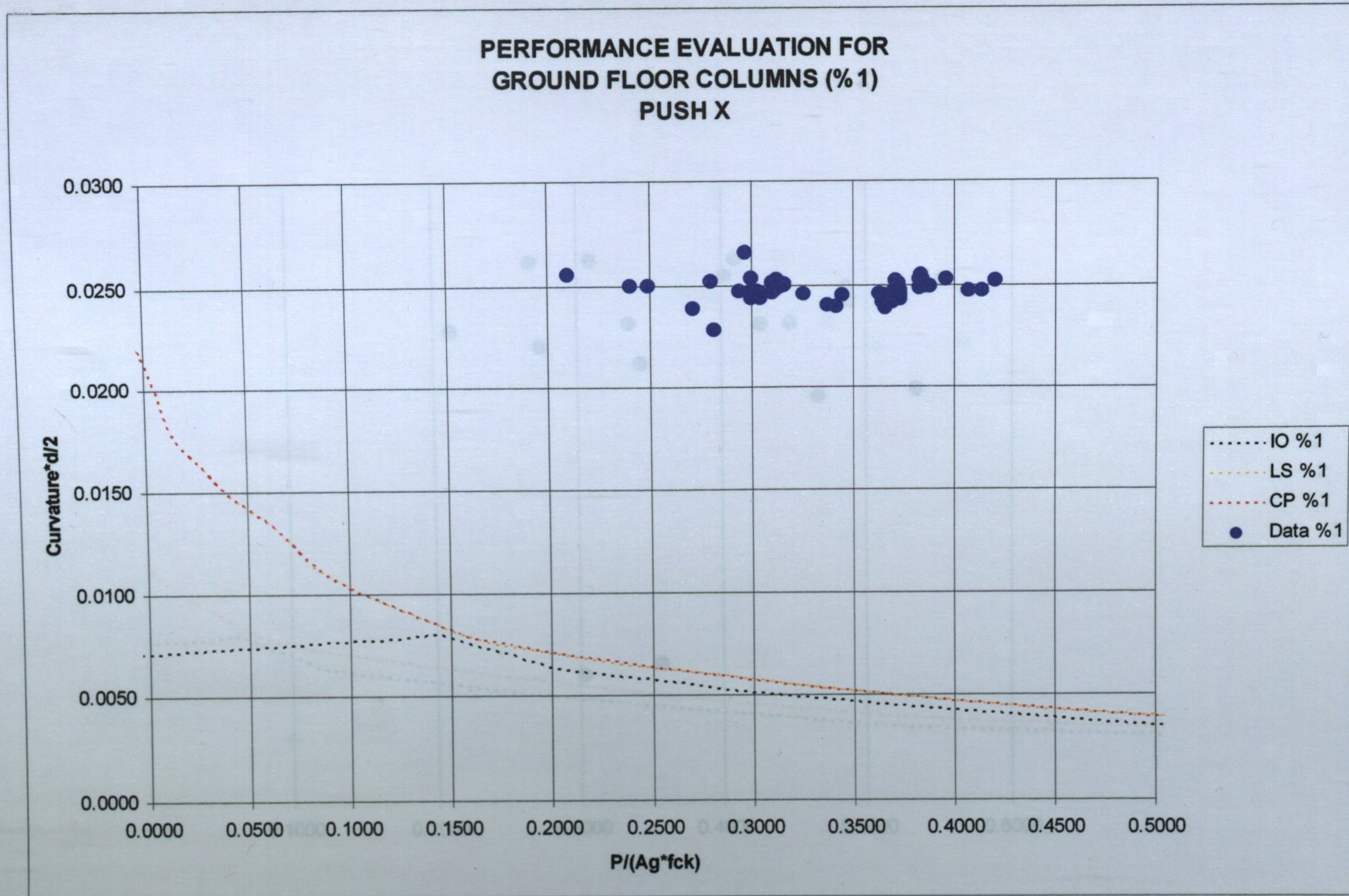
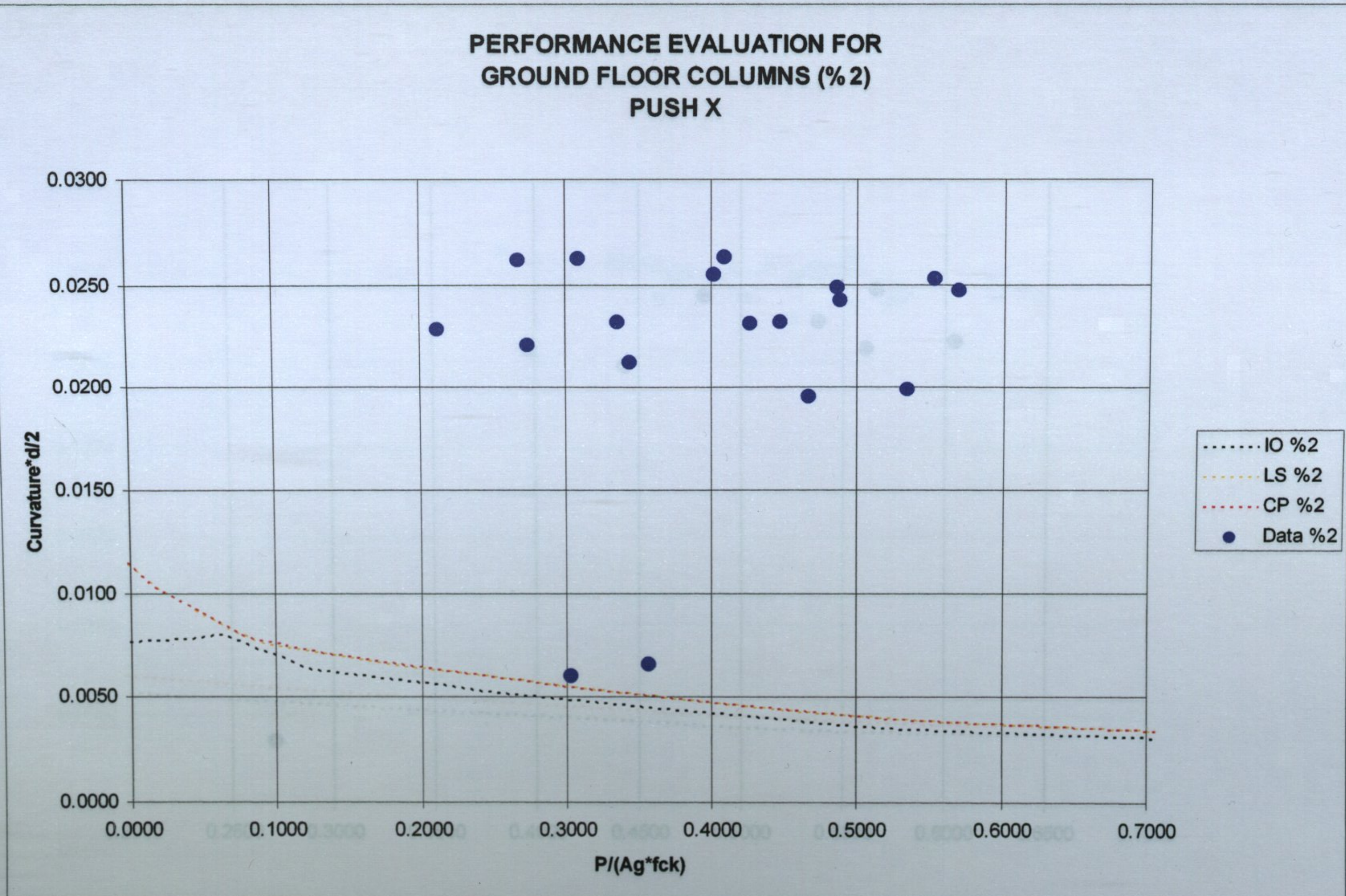
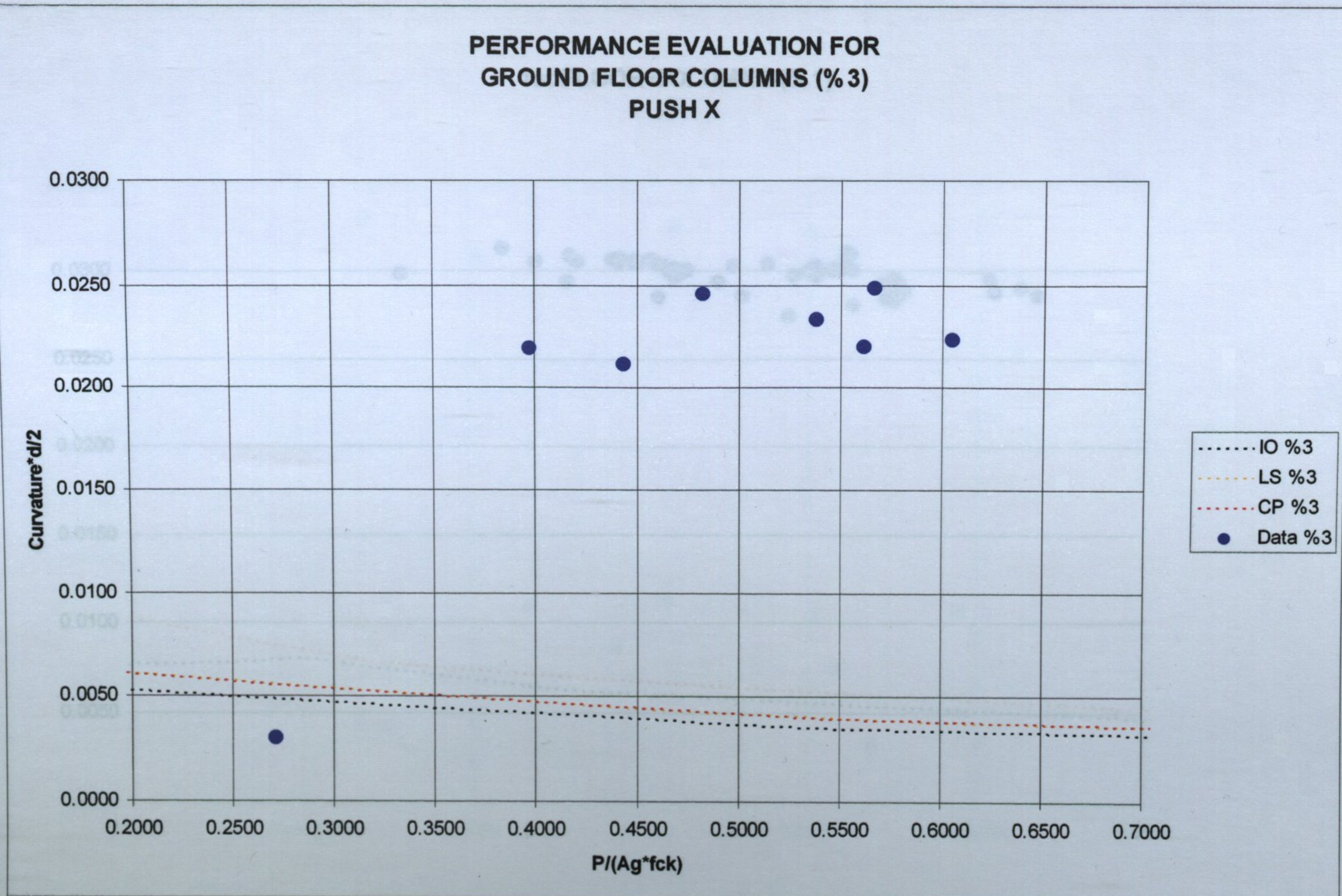


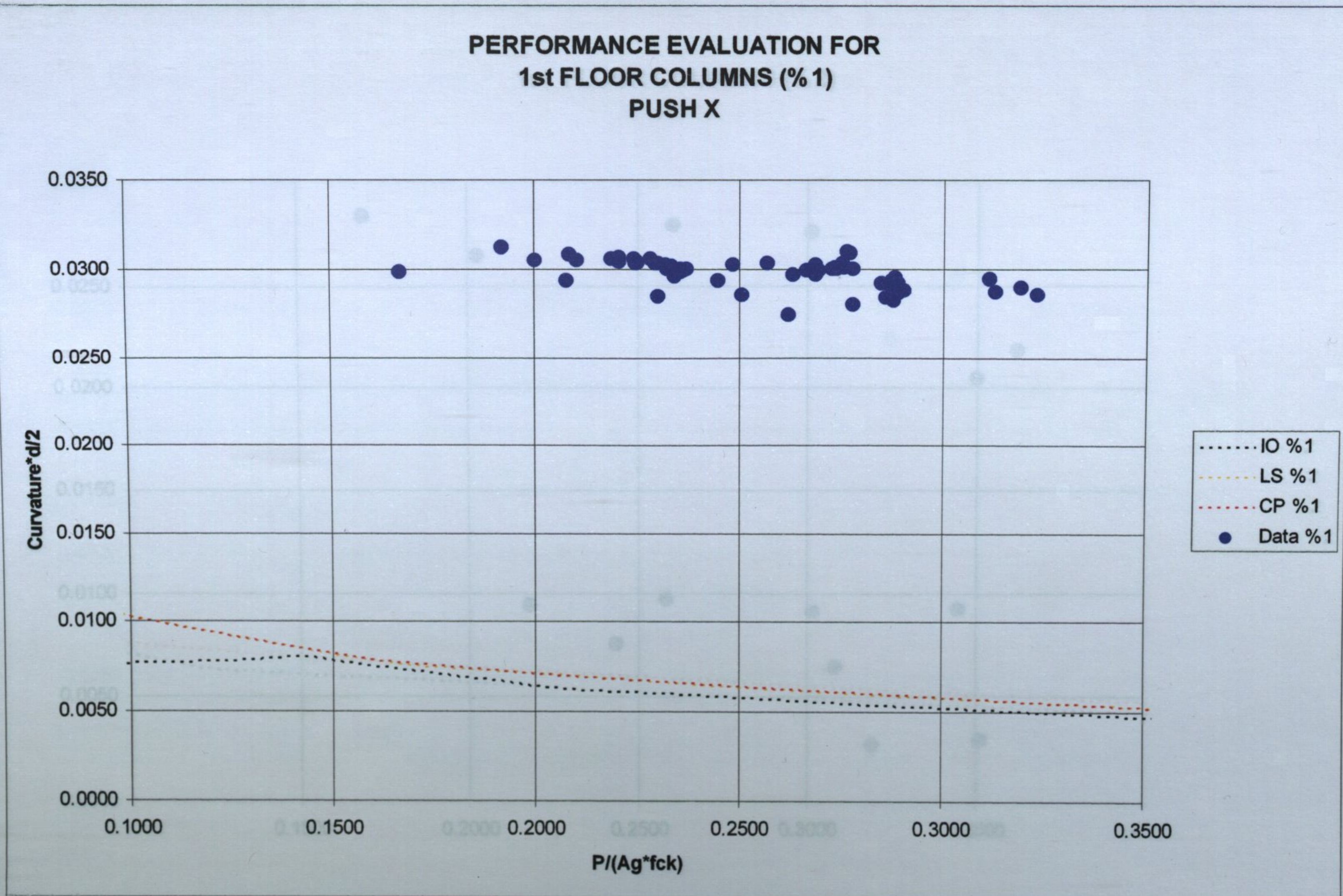
Figure 4.8. Hinge locations for X directional pushover analysis

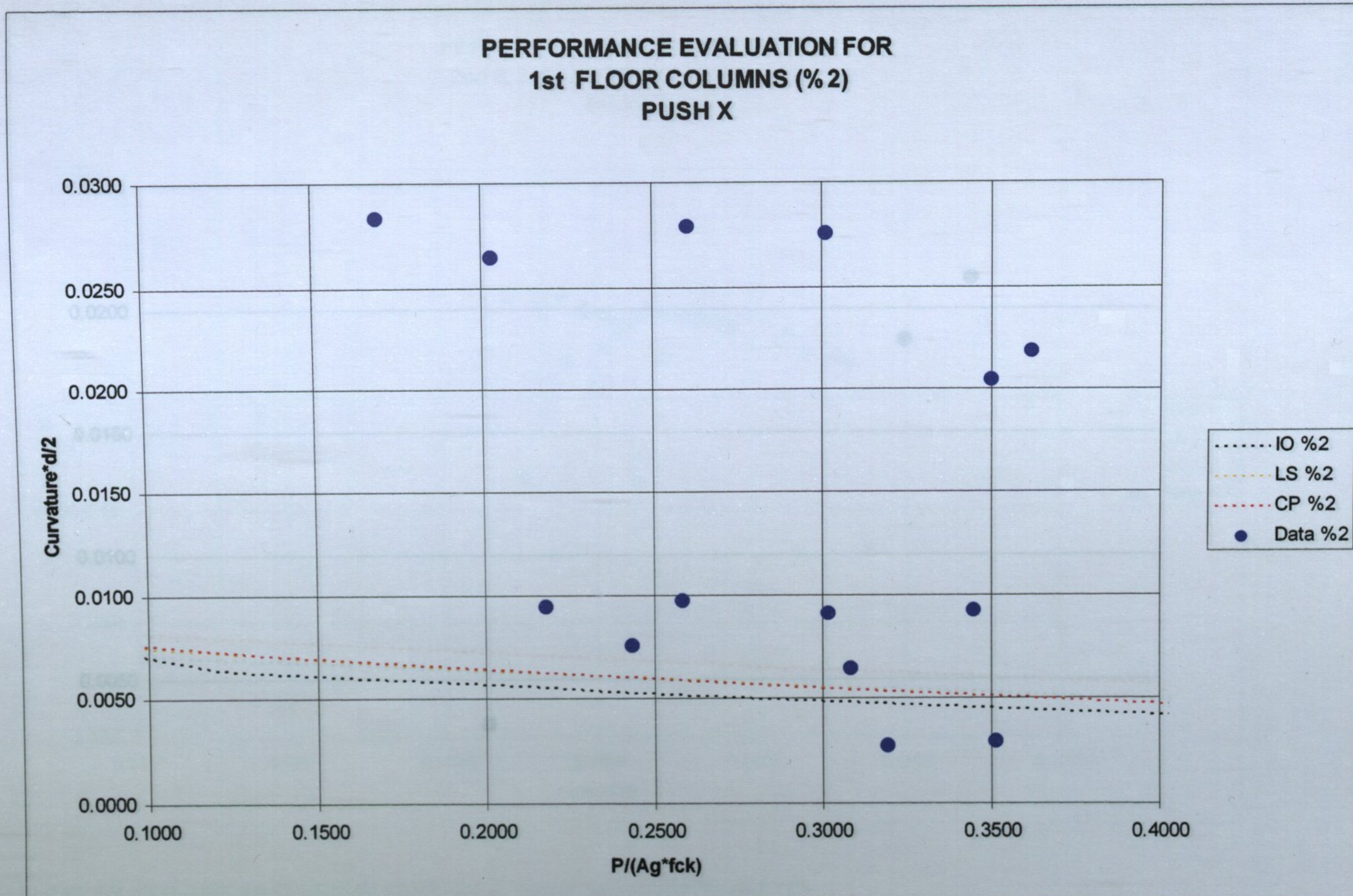

PERFORMANCE EVALUATION FOR
GROUND FLOOR COLUMNS (%1)
PUSH X
Curvature*d/2
0.0300
0.0250
0.0200
0.0150
0.0100
0.0050
0.0000
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
0.3500
0.4000
0.4500
0.5000
P/(Ag*fck)
IO %1
LS %1
CP %1
Data %1

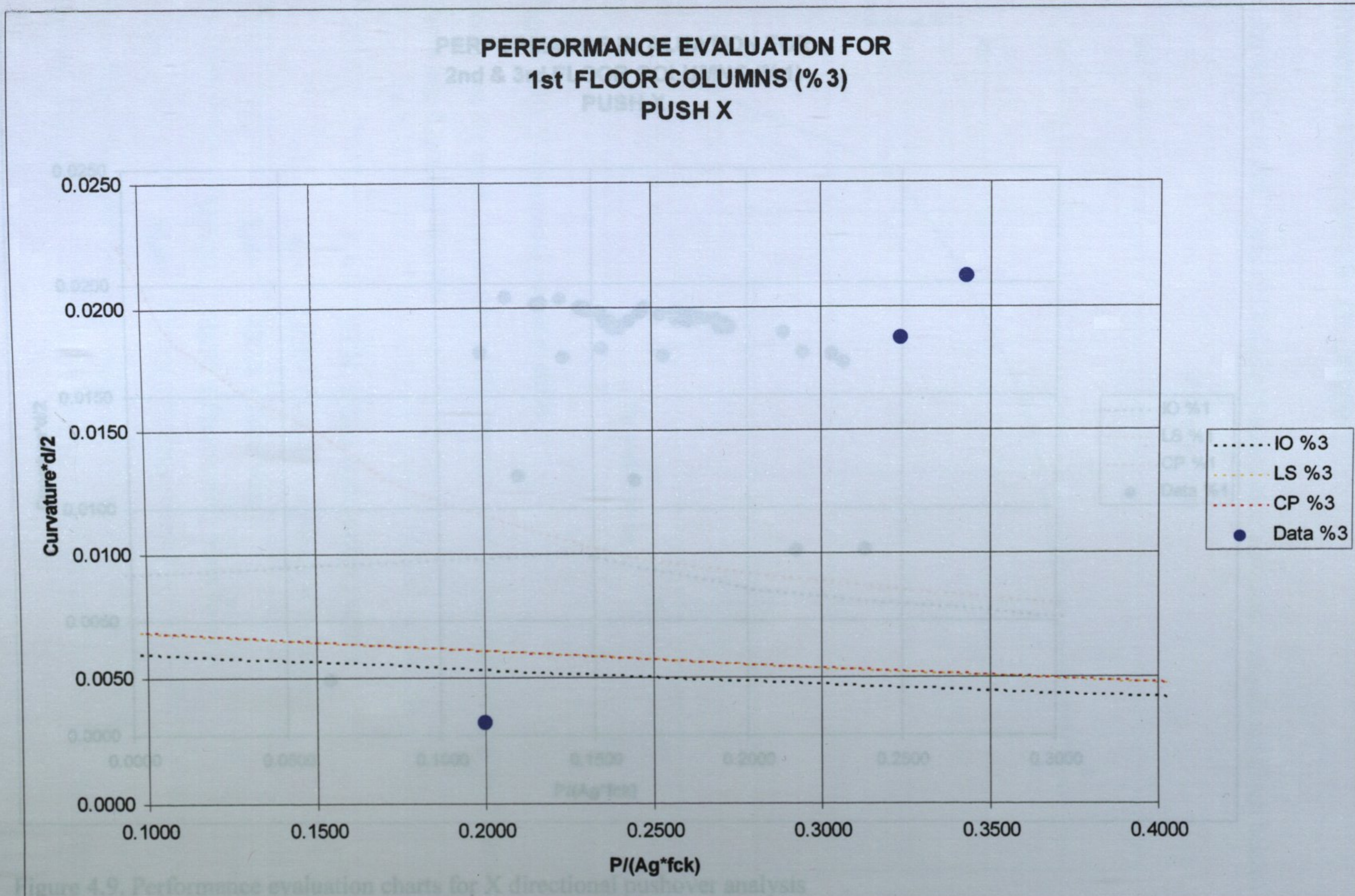

PERFORMANCE EVALUATION FOR
GROUND FLOOR COLUMNS (%2)
PUSH X
Curvature*d/2
0.0300
0.0250
0.0200
0.0150
0.0100
0.0050
0.0000
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
0.6000
0.7000
P/(Ag*fck)
IO %2
LS %2
CP %2
Data %2



PERFORMANCE EVALUATION FOR
GROUND FLOOR COLUMNS (%3)
PUSH X
Curvature*d/2
0.0300
0.0250
0.0200
0.0150
0.0100
0.0050
0.0000
0.2000
0.2500
0.3000
0.3500
0.4000
0.4500
0.5000
0.5500
0.6000
0.6500
0.7000
P/(Ag*fck)
IO %3
LS %3
CP %3
Data %3


PERFORMANCE EVALUATION FOR
1st FLOOR COLUMNS (%1)
PUSH X
Curvature*d/2
0.0350
0.0300
0.0250
0.0200
0.0150
0.0100
0.0050
0.0000
0.1000
0.1500
0.2000
0.2500
0.3000
0.3500
P/(Ag*fck)
IO %1
LS %1
CP %1
Data %1


PERFORMANCE EVALUATION FOR
1st FLOOR COLUMNS (%2)
PUSH X
Curvature*d/2
0.0300
0.0250
0.0200
0.0150
0.0100
0.0050
0.0000
0.1000
0.1500
0.2000
0.2500
0.3000
0.3500
0.4000
P/(Ag*fck)
IO %2
LS %2
CP %2
Data %2


PERFORMANCE EVALUATION FOR
1st FLOOR COLUMNS (%3)
PUSH X
Curvature*d/2
0.0250
0.0200
0.0150
0.0100
0.0050
0.0000
0.1000
0.1500
0.2000
0.2500
0.3000
0.3500
0.4000
P/(Ag*fck)
IO %3
LS %3
CP %3
Data %3

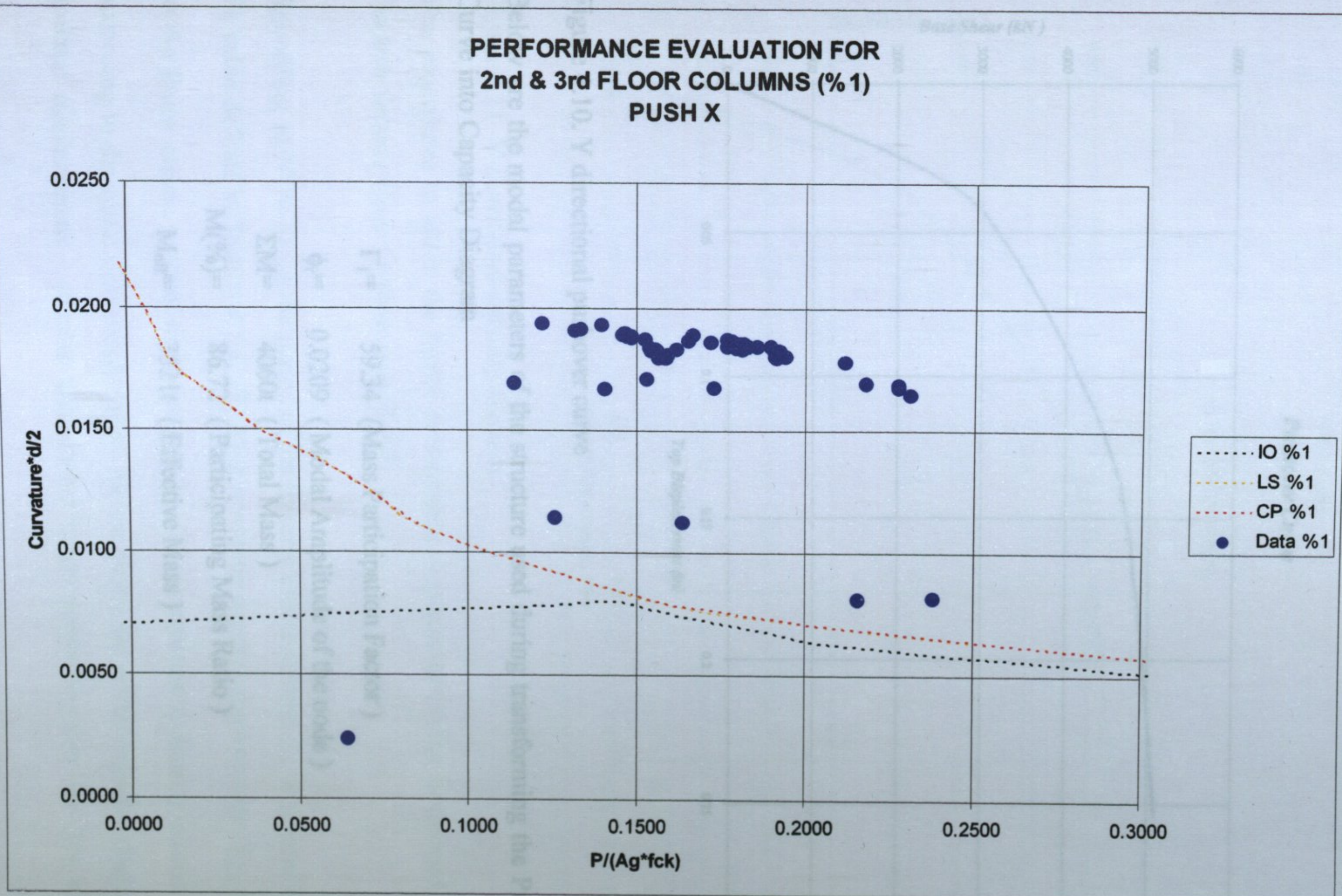


Figure 4.9. Performance evaluation charts for X directional pushover analysis

Pushover Analysis Results for the Direction Y

Pushover Curve and Capacity Diagram obtained after nonlinear analysis are given below.

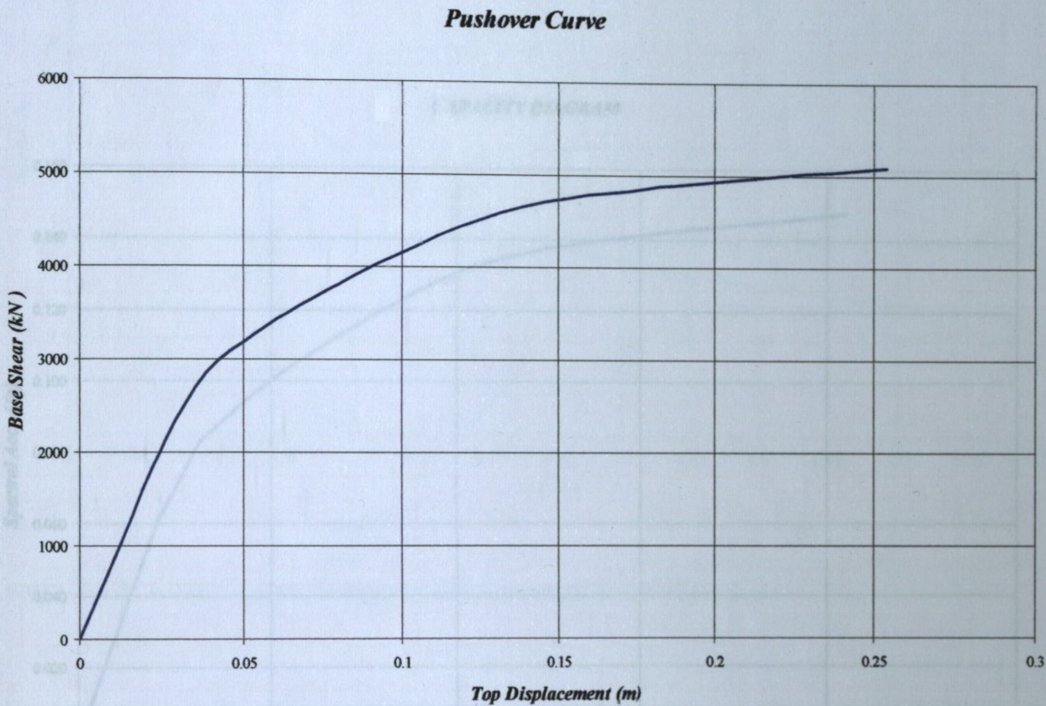


Figure 4.10. Y directional pushover curve

Below are the modal parameters of the structure used during transforming the Pushover Curve into Capacity Diagram

Γ_1= 59.34 (Mass Participation Factor)

ϕ_r= 0.0209 (Modal Amplitude of the node)

ΣM= 4060t (Total Mass)

M(%)= 86.72 (Participating Mass Ratio)

M_{eff}= 3521t (Effective Mass)

And the circular frequency and period of the system using the cracked stiffness properties with the modal parameters given above are as below.

$\omega^2 =$ 29.28 rad/sec

T= 1.161 sec

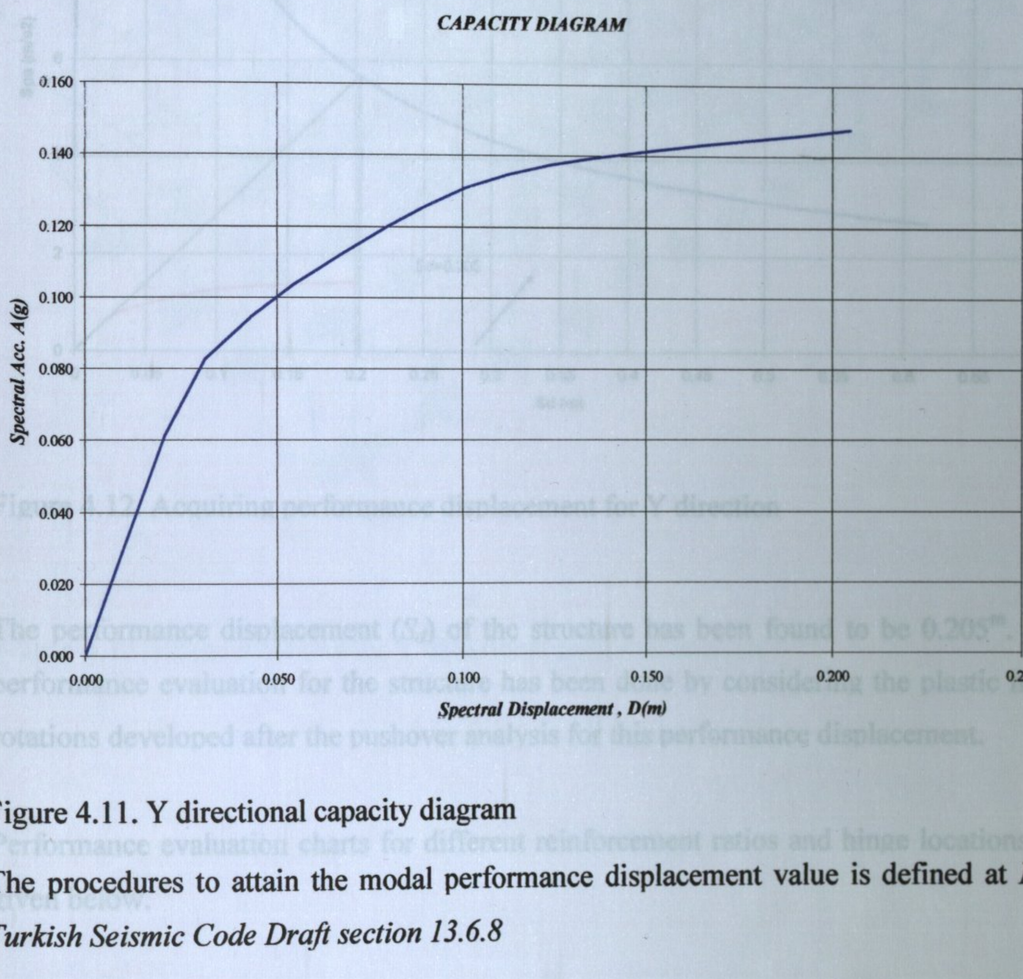


Figure 4.11. Y directional capacity diagram

The procedures to attain the modal performance displacement value is defined at *New Turkish Seismic Code Draft section 13.6.8*

According to NTSCD, as the period of the linearly elastic system T_1, is greater than the T_B value defined in *Section 4.1.4.* $(T_1 \geq T_B)$, nonlinear spectral displacement $S_{di,1}$ is equal to the linear elastic spectral displacement $S_{de,1}$ of the equivalent linearly elastic system according to the equal displacement rule. Below charts are modal capacity diagram and spectral displacement – spectral acceleration charts drawn together for this example.

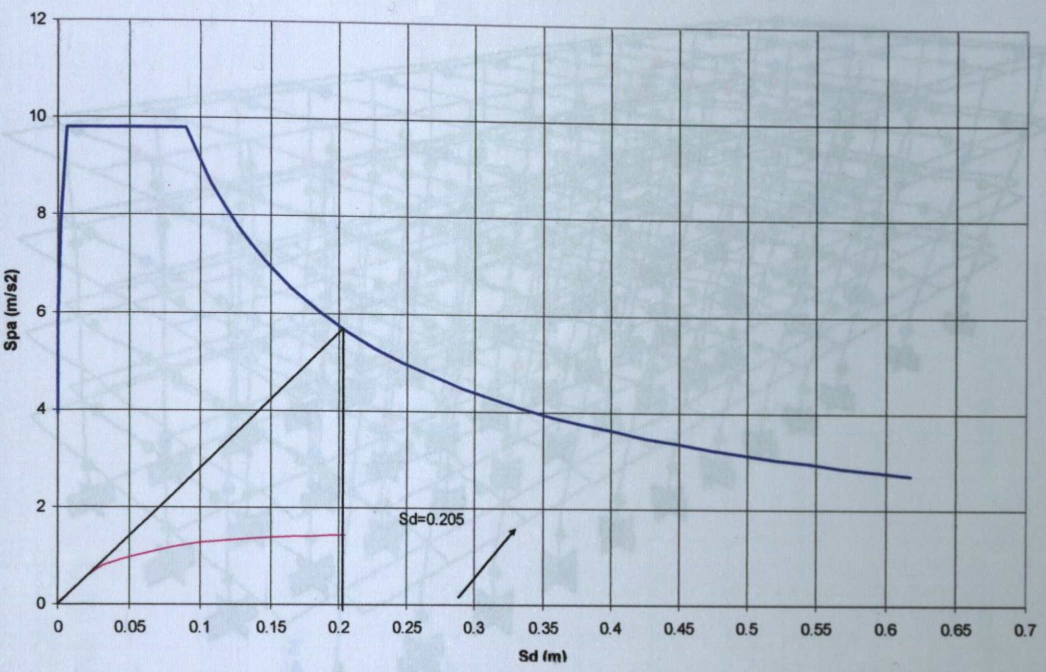


Figure 4.12. Acquiring performance displacement for Y direction

The performance displacement (S_d) of the structure has been found to be 0.205^{m}. And performance evaluation for the structure has been done by considering the plastic hinge rotations developed after the pushover analysis for this performance displacement.

Performance evaluation charts for different reinforcement ratios and hinge locations are given below.

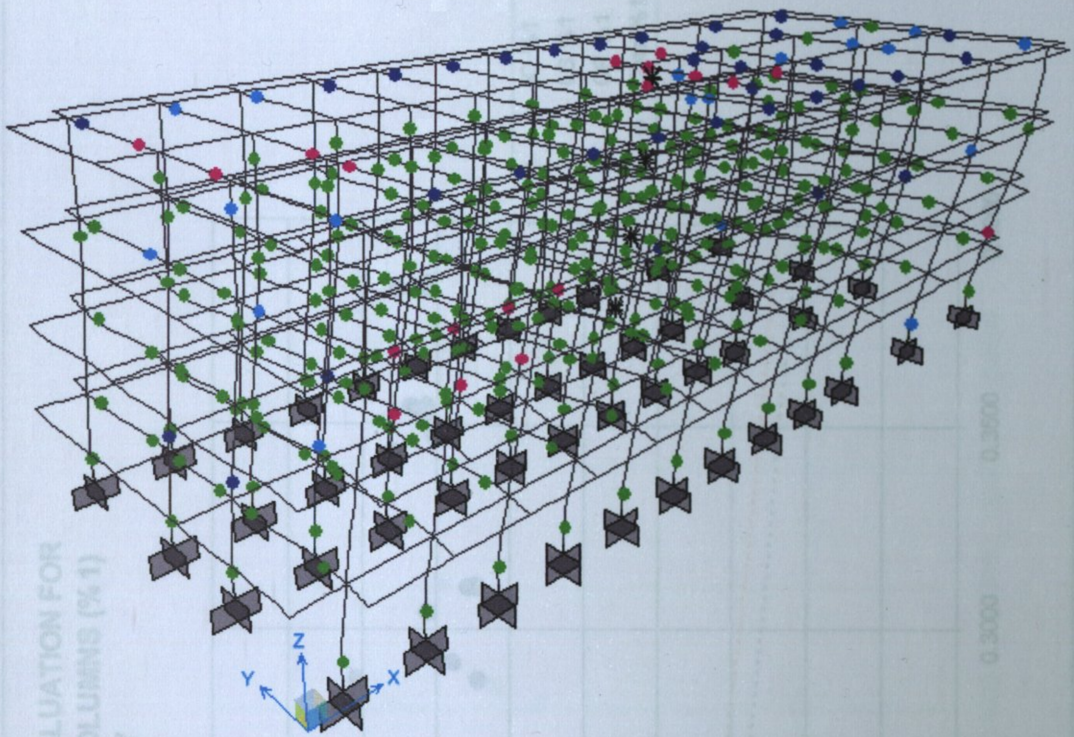
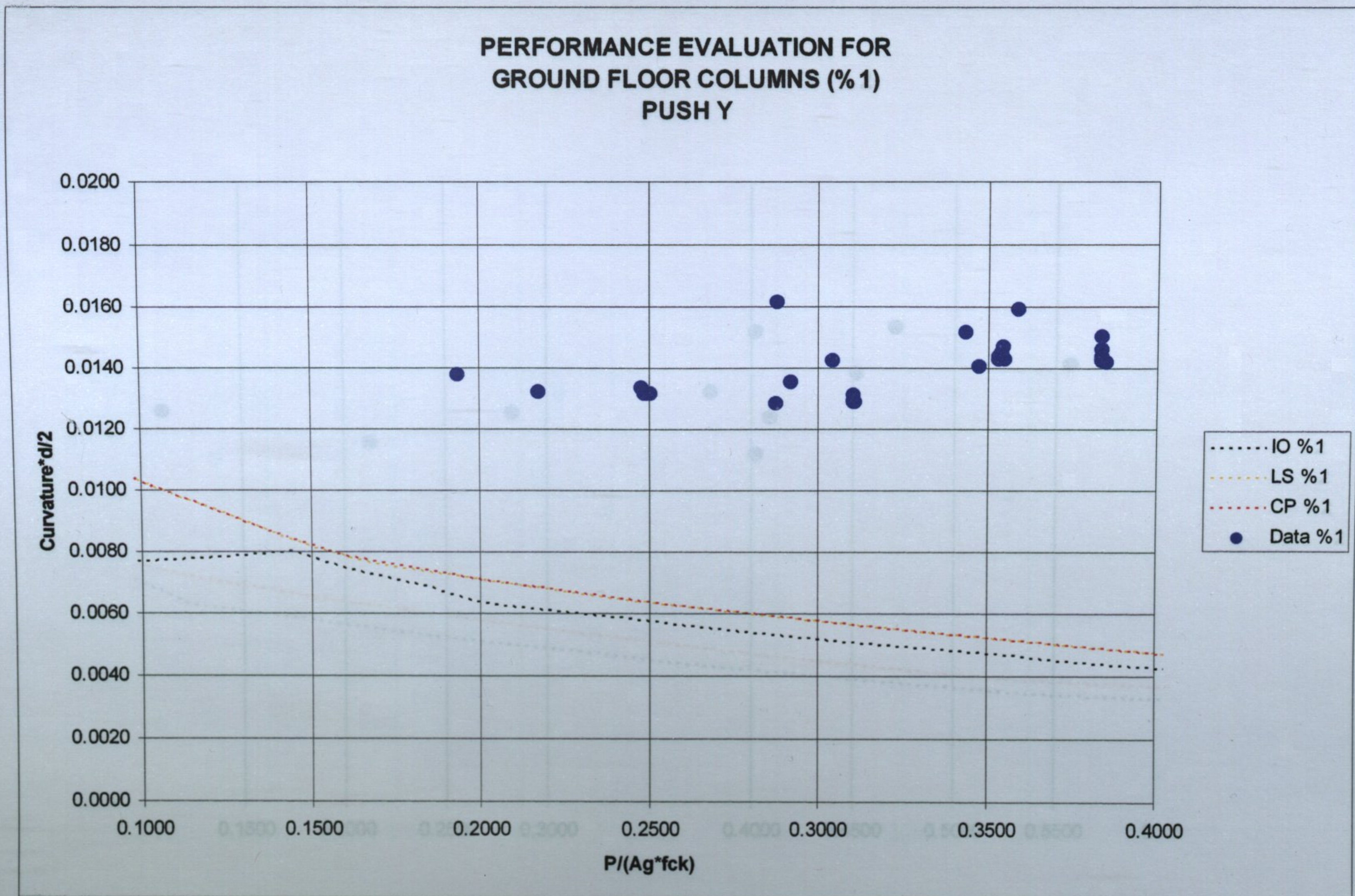
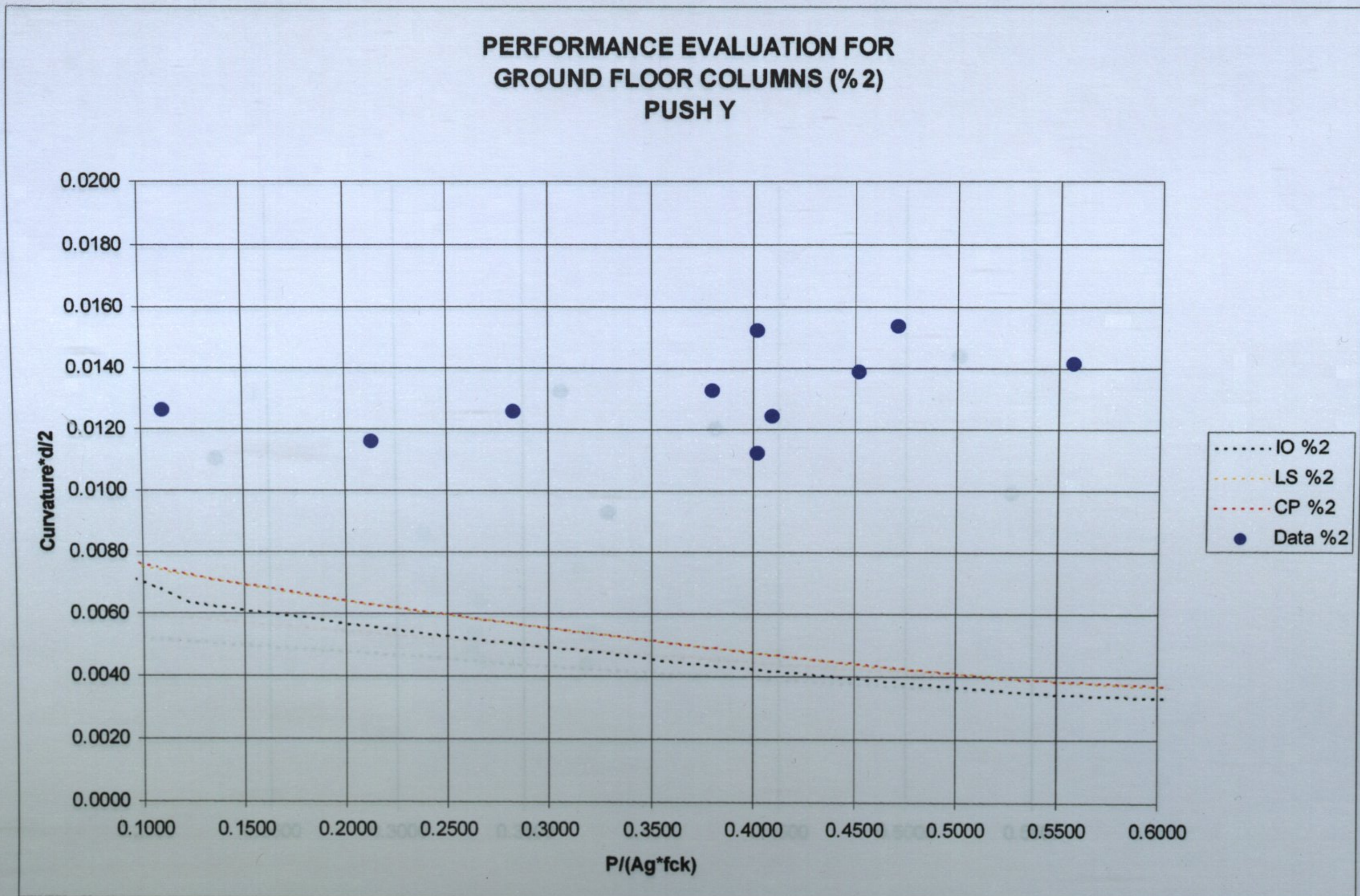
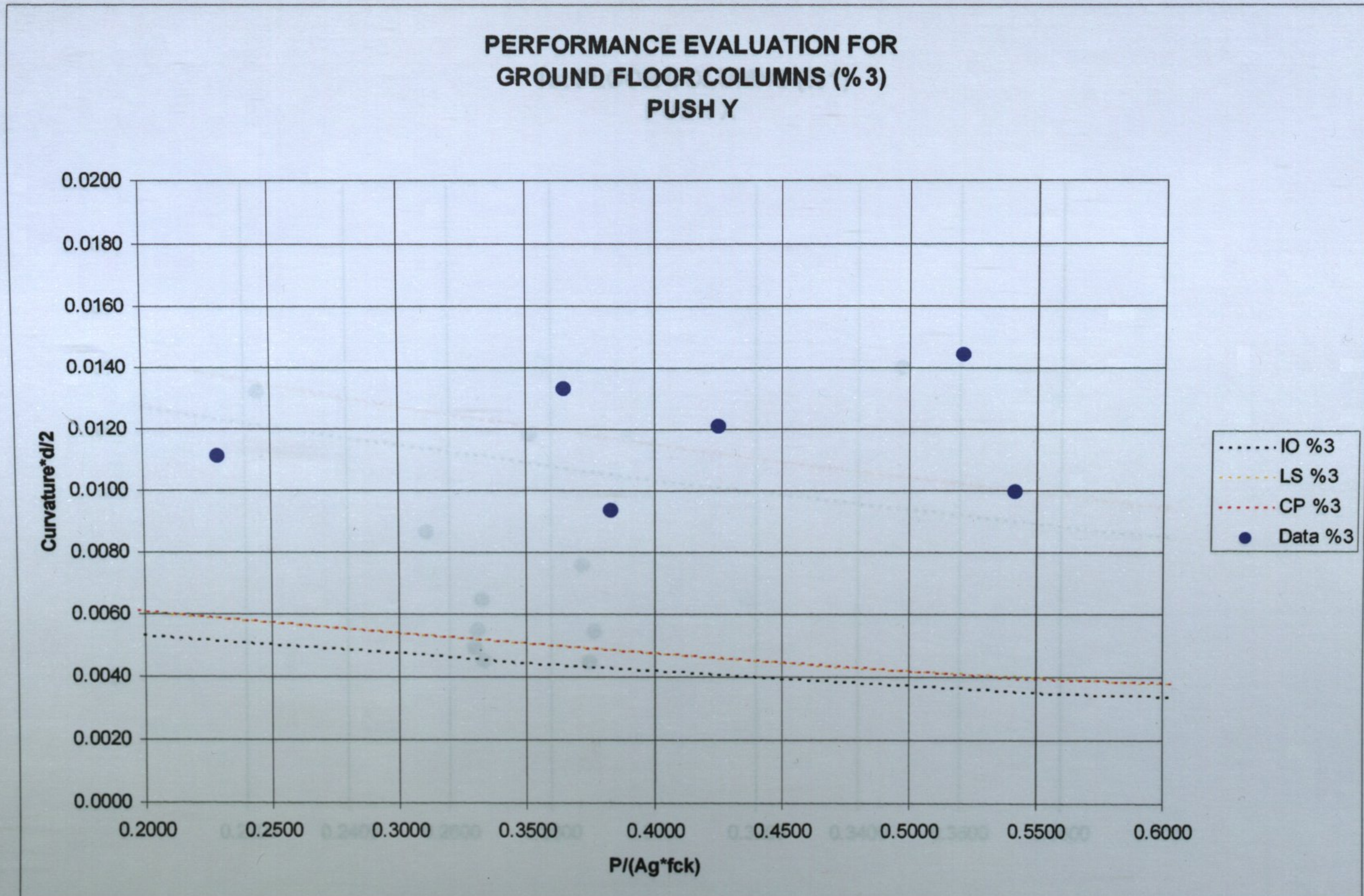
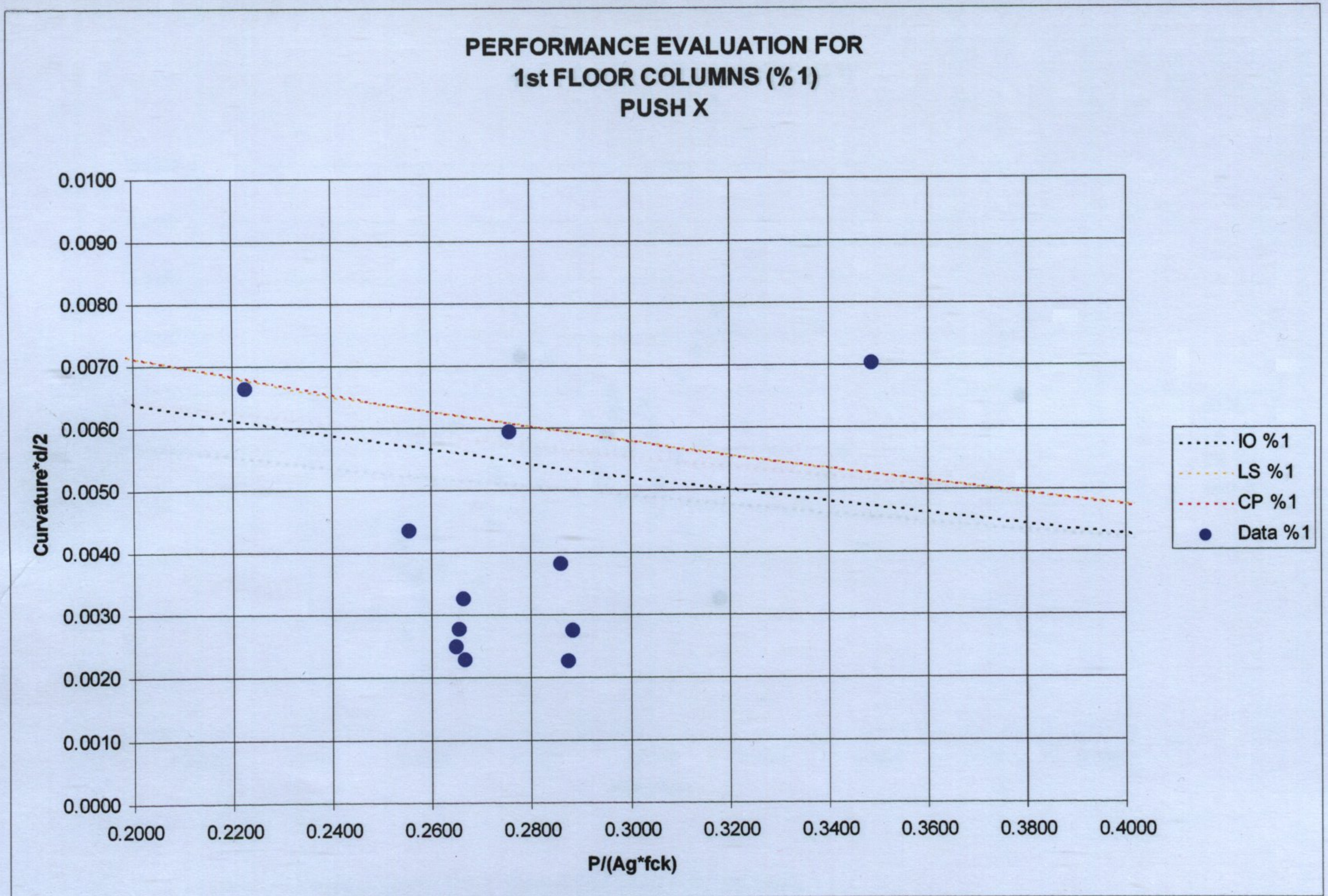


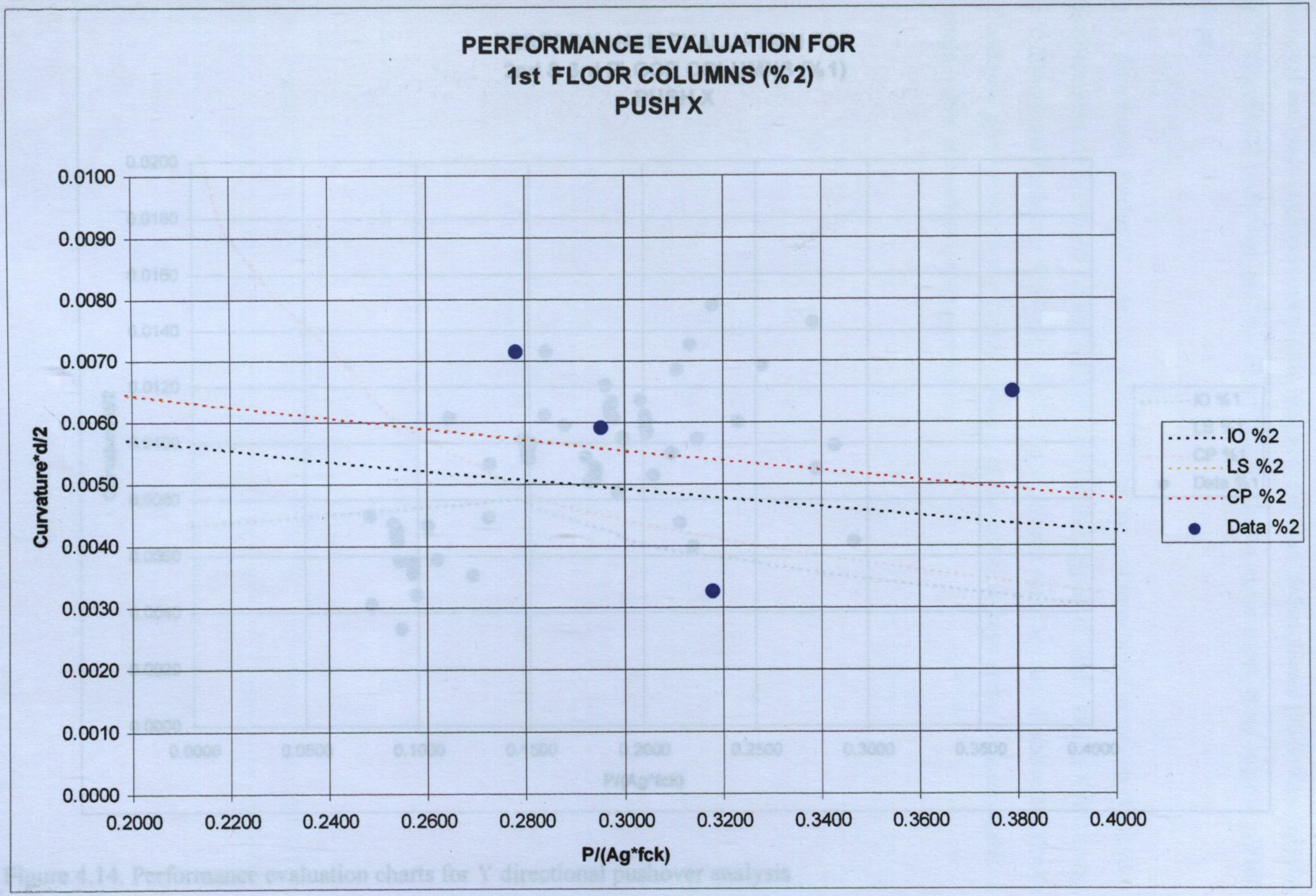
Figure 4.13. Hinge locations for Y directional pushover analysis


PERFORMANCE EVALUATION FOR
GROUND FLOOR COLUMNS (%1)
PUSH Y
Curvature*d/2
0.0200
0.0180
0.0160
0.0140
0.0120
0.0100
0.0080
0.0060
0.0040
0.0020
0.0000
0.1000
0.1500
0.2000
0.2500
0.3000
0.3500
0.4000
P/(Ag*fck)
IO %1
LS %1
CP %1
Data %1


PERFORMANCE EVALUATION FOR
GROUND FLOOR COLUMNS (% 2)
PUSH Y
Curvature*d/2
P/(Ag*fck)
0.0200
0.0180
0.0160
0.0140
0.0120
0.0100
0.0080
0.0060
0.0040
0.0020
0.0000
0.1000
0.1500
0.2000
0.2500
0.3000
0.3500
0.4000
0.4500
0.5000
0.5500
0.6000
IO %2
LS %2
CP %2
Data %2


PERFORMANCE EVALUATION FOR
GROUND FLOOR COLUMNS (% 3)
PUSH Y
Curvature*d/2
0.0200
0.0180
0.0160
0.0140
0.0120
0.0100
0.0080
0.0060
0.0040
0.0020
0.0000
0.2000
0.2500
0.3000
0.3500
0.4000
0.4500
0.5000
0.5500
0.6000
P/(Ag*fck)
IO %3
LS %3
CP %3
Data %3


PERFORMANCE EVALUATION FOR
1st FLOOR COLUMNS (%1)
PUSH X
Curvature*d/2
0.0100
0.0090
0.0080
0.0070
0.0060
0.0050
0.0040
0.0030
0.0020
0.0010
0.0000
0.2000
0.2200
0.2400
0.2600
0.2800
0.3000
0.3200
0.3400
0.3600
0.3800
0.4000
P/(Ag*fck)
IO %1
LS %1
CP %1
Data %1


PERFORMANCE EVALUATION FOR
1st FLOOR COLUMNS (%2)
PUSH X
Curvature*d/2
0.0100
0.0090
0.0080
0.0070
0.0060
0.0050
0.0040
0.0030
0.0020
0.0010
0.0000
0.2000
0.2200
0.2400
0.2600
0.2800
0.3000
0.3200
0.3400
0.3600
0.3800
0.4000
P/(Ag*fck)
IO %2
LS %2
CP %2
Data %2

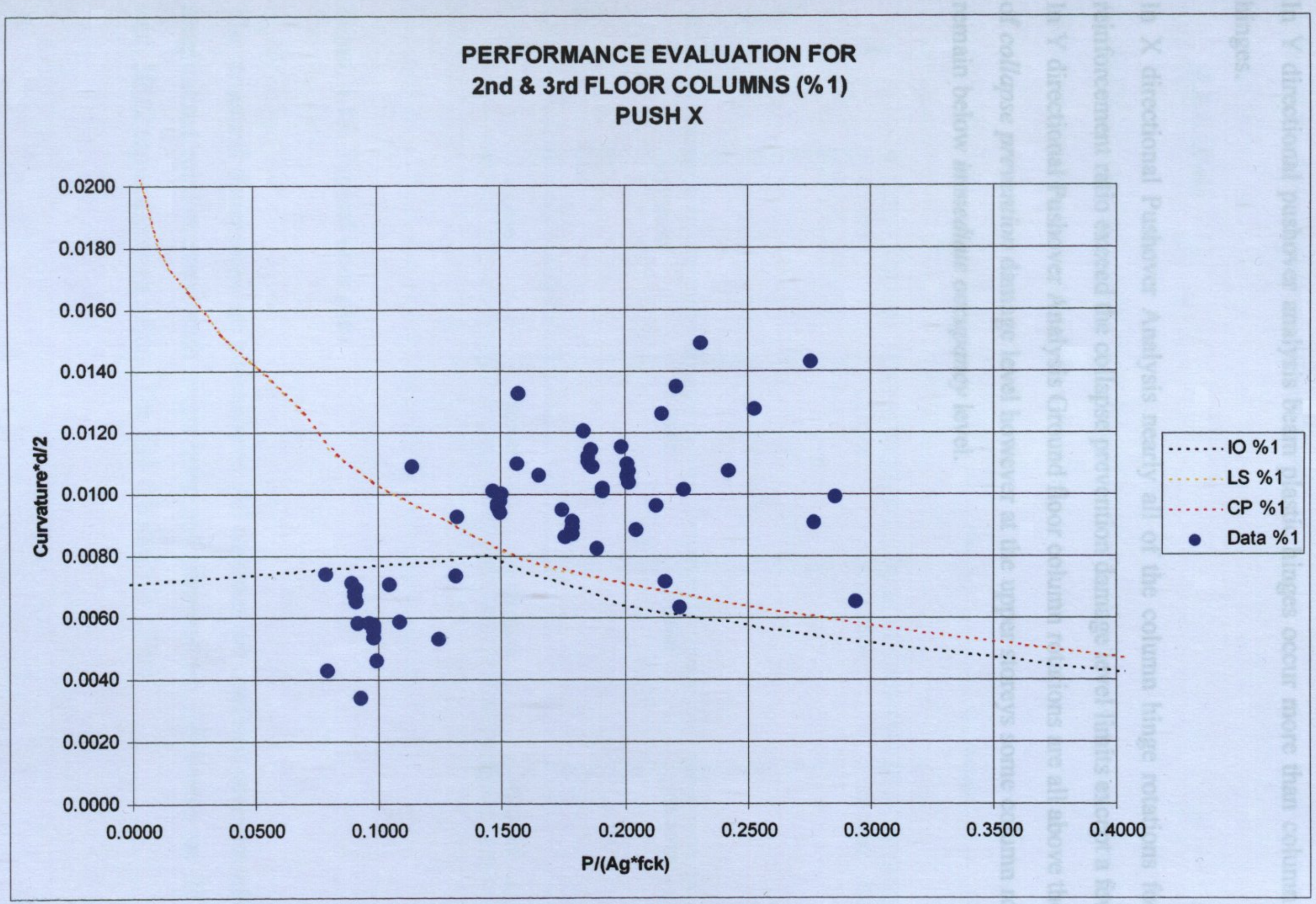


Figure 4.14. Performance evaluation charts for Y directional pushover analysis

Exact difference between the X and Y direction pushover analysis is that in X directional pushover analysis more column plastic hinges occur than Y directional pushover analysis. In Y directional pushover analysis beam plastic hinges occur more than column plastic hinges.

In X directional Pushover Analysis nearly all of the column hinge rotations for every reinforcement ratio exceed the collapse prevention damage level limits except a few ones. In Y directional Pushover Analysis Ground floor column rotations are all above the limits of *collapse prevention* damage level however at the upper storeys some column rotations remain below *immediate occupancy* level.

4.2. Performance Evaluation of a Recently Well Designed Building Using Strain Based Performance Evaluation Charts

4.2.1. Data

Performance of a four storey Reinforced Concrete frame building has been evaluated with Pushover Analysis. All of the storey heights are 3m.

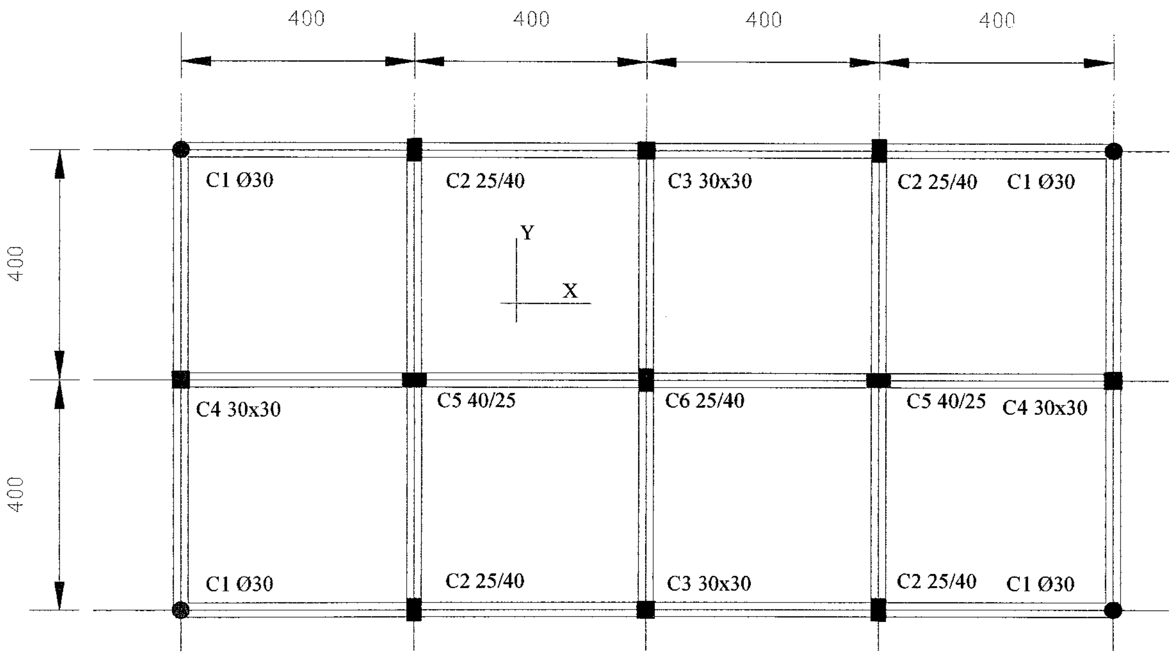


Figure 4.15. Typical floor plan

The structural dimensions are given above. All columns are confined concrete with 8Ø14 longitudinal reinforcement with single hoop and diamond tie. All beams are 25^{cm}x40^{cm} with 3Ø12 top and bottom rebars. The slab thickness is 12 cm.

4.2.2. Materials

<u>*Concrete*</u>

fck = 25 MPa

Ec= $3250\sqrt{(fck)} + 14000$ MPa

Ec= $3250\sqrt{25} + 14000 = 30250$MPa = 3.025 E7 kN/m2

<u>*Steel*</u>

Longitudinal Steel: S420 $f_{yk} = 420$ MPa

Transverse Steel : S420 $f_{yk} = 420$ MPa

4.2.3. Effecting Vertical Loads

Reinforced Concrete Slab	0.12*25.00	= 3.00 kN/m^2
Superimposed Dead		= 1.50 kN/m^2
Live		= 2.00 kN/m^2

4.2.4. Elastic Response Spectrum

Elastic Response Spectrum given in *TSC98* has been considered at *1st Seismic Zone* with building importance factor *I=1.0* and T_a, T_b values given below

T_A= 0.15 sec.

T_B=0.60 sec.

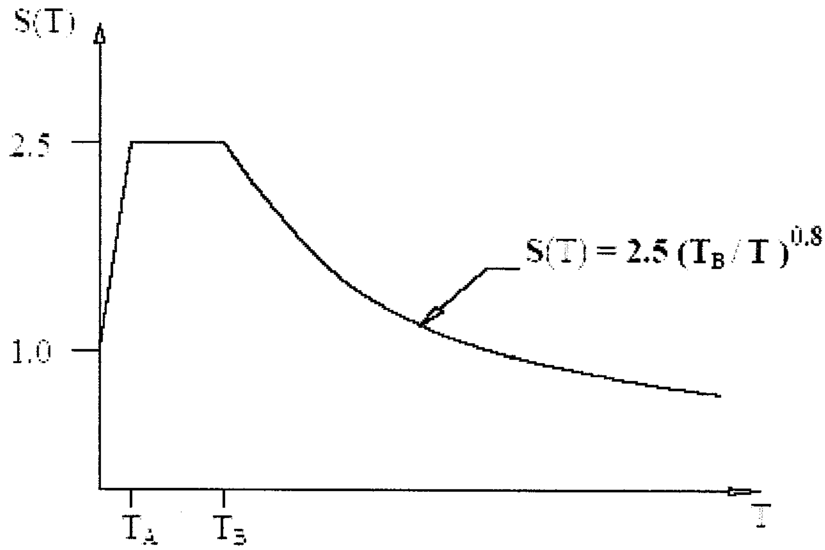


Figure 4.16. Elastic response spectrum (TSC98)

4.2.5. Non-Linear Static Pushover Analysis of the Structure

The structure has been modeled using a nonlinear structural analysis software '*SAP2000*'. Plastic hinge properties have been defined by considering column '*axial force – bending moment*' and beam '*bending moment – curvature*' relations and then these properties have been assigned to the related beam and column end points.

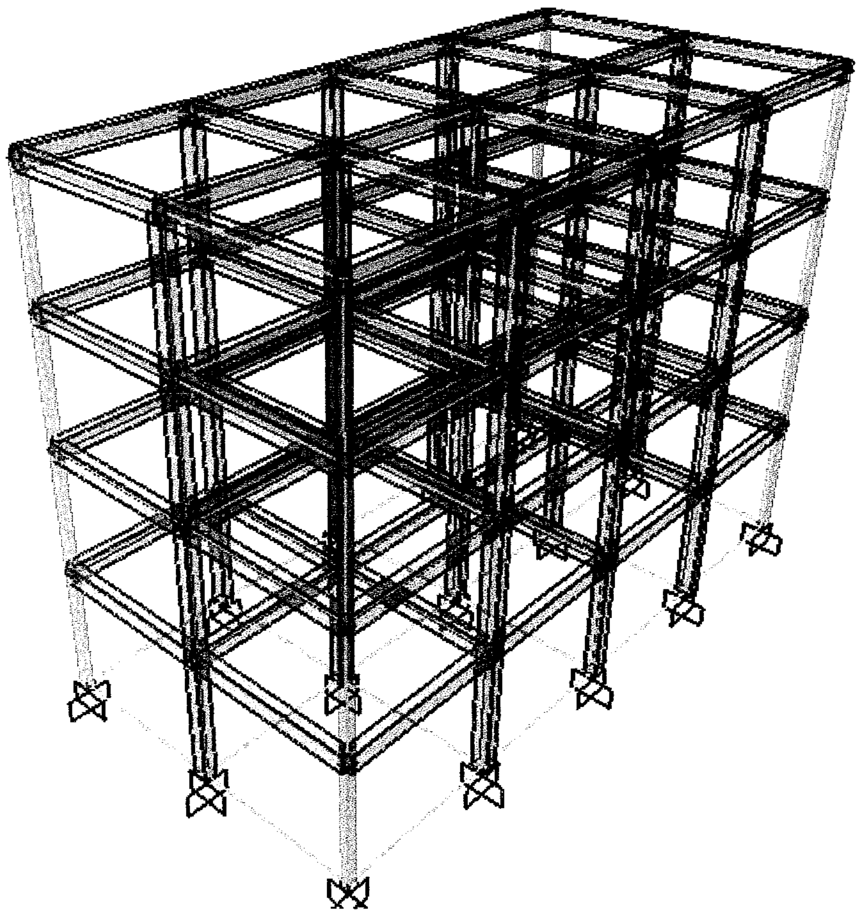


Figure 4.17. 3D model of the structure

Pushover Analysis Results for the Direction X

Pushover Curve and Capacity Diagram obtained after nonlinear analysis are given below.

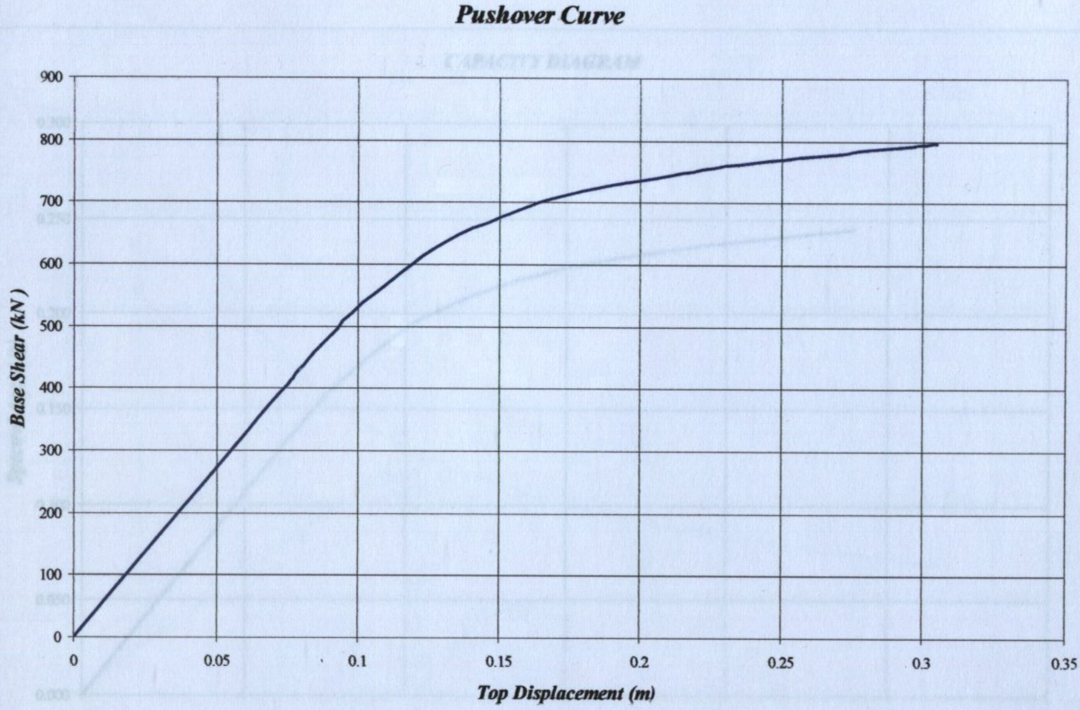


Figure 4.18. X directional pushover curve

Below are the modal parameters of the structure used during transforming the Pushover Curve into Capacity Diagram.

Γ_1= -18.208 (Mass Participation Factor)

ϕ_r= -0.0697 (Modal Amplitude of the node)

ΣM= 391.82t (Total Mass)

M(%)= 84.6 (Participating Mass Ratio)

M_{eff}= 331t (Effective Mass)

And the circular frequency and period of the system using the cracked stiffness properties with the modal parameters given above are as below.

ω^2= 20.82 rad/sec

T= 1.377 sec

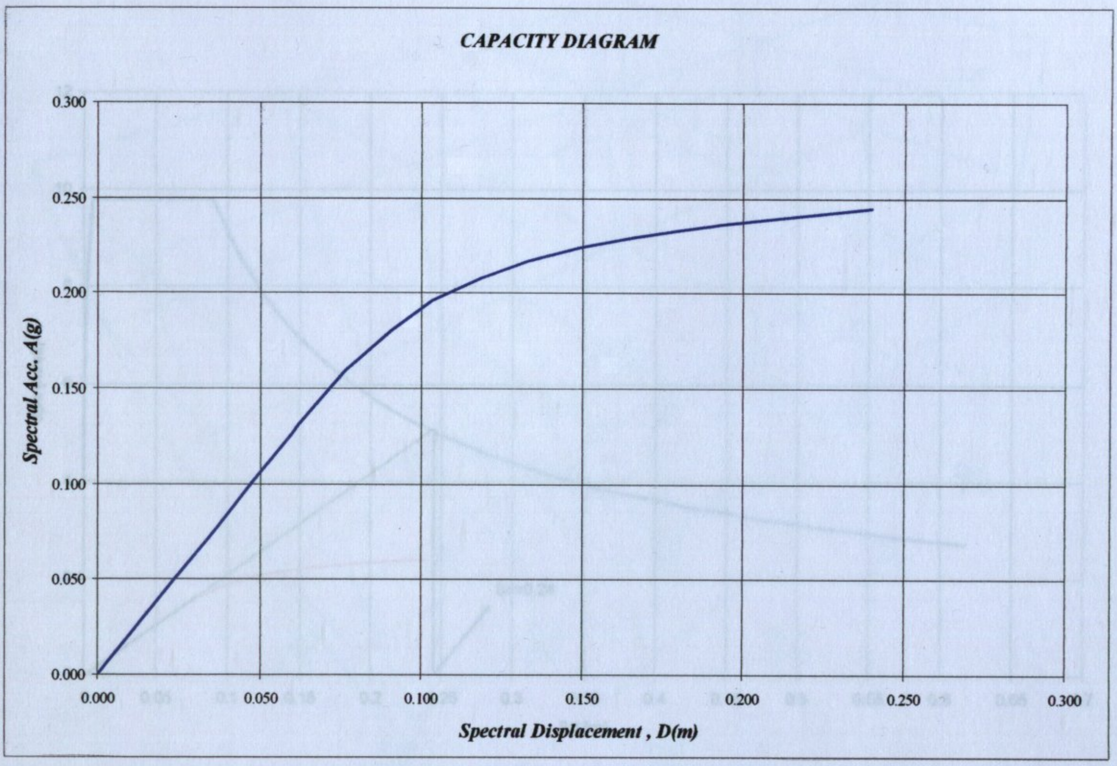


Figure 4.19. X directional capacity diagram

The procedures to attain the modal performance displacement value are defined at *New Turkish Seismic Code Draft section 13.6.8*

According to *New Turkish Seismic Code Draft*, as the period of the linearly elastic system T_1, is greater than the T_B value defined in *Section 4.1.4.* $(T_1 \geq T_B)$, nonlinear spectral displacement $S_{di,1}$ is equal to the linear elastic spectral displacement $S_{de,1}$ of the equivalent linearly elastic system according to the equal displacement rule. Below charts are modal capacity diagram and spectral displacement – spectral acceleration charts drawn together for this example.

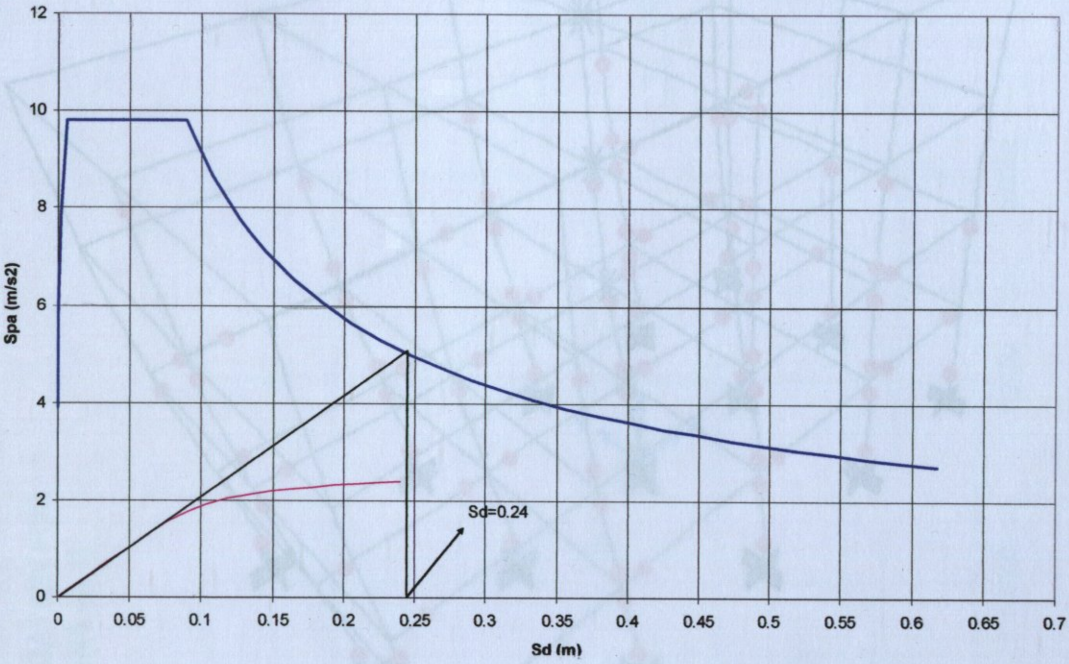


Figure 4.20. Acquiring performance displacement for X direction

The performance displacement (S_d) of the structure has been found to be 0.24^m. And performance evaluation for the structure has been done by considering the plastic hinge rotations developed after the pushover analysis for this performance displacement.

Performance evaluation charts for different reinforcement ratios and hinge locations are given below.

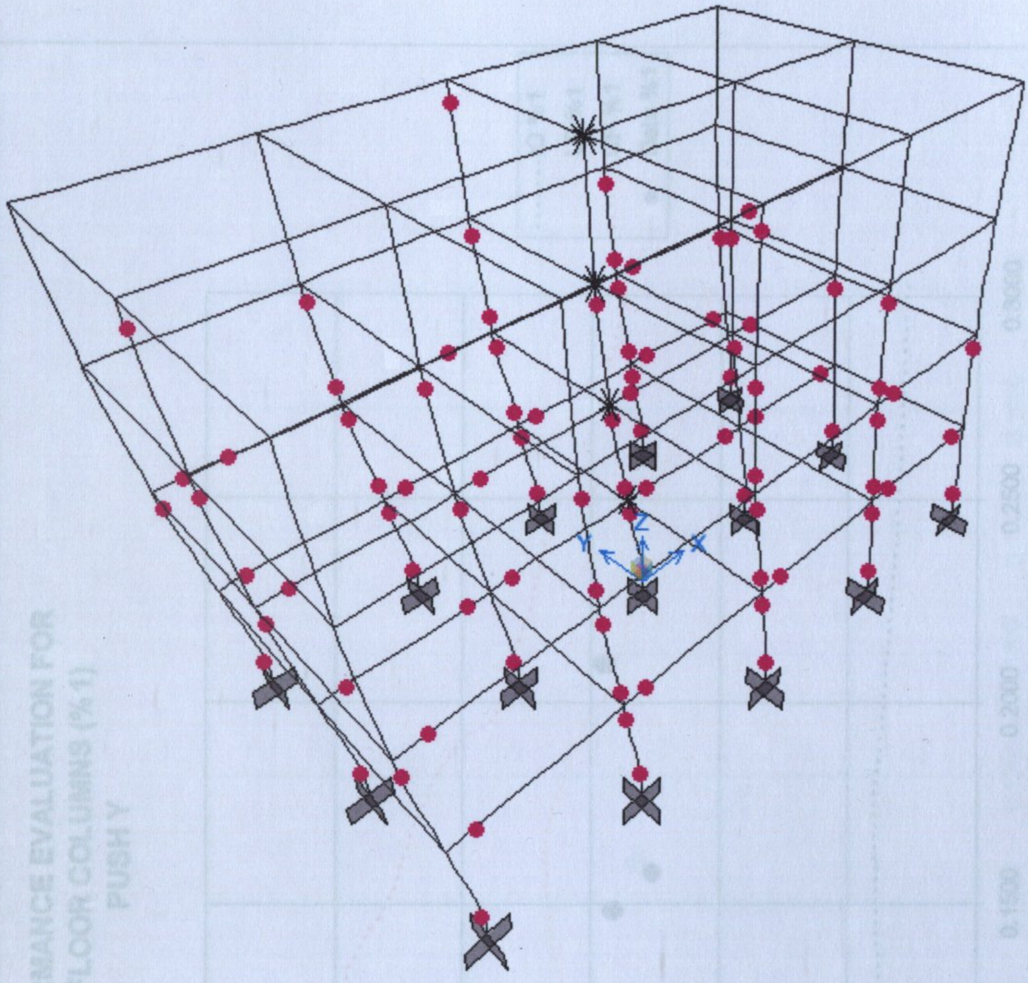
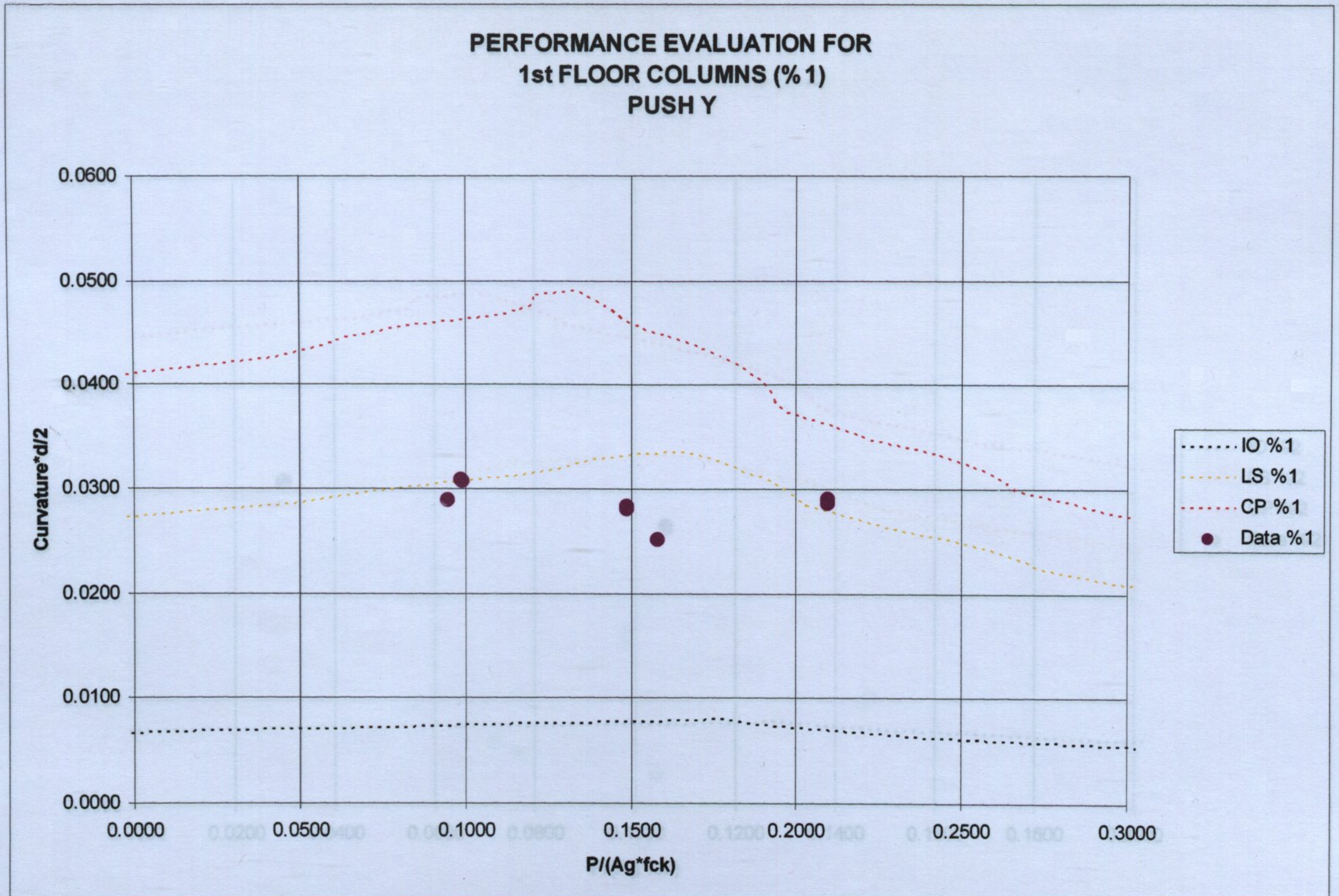
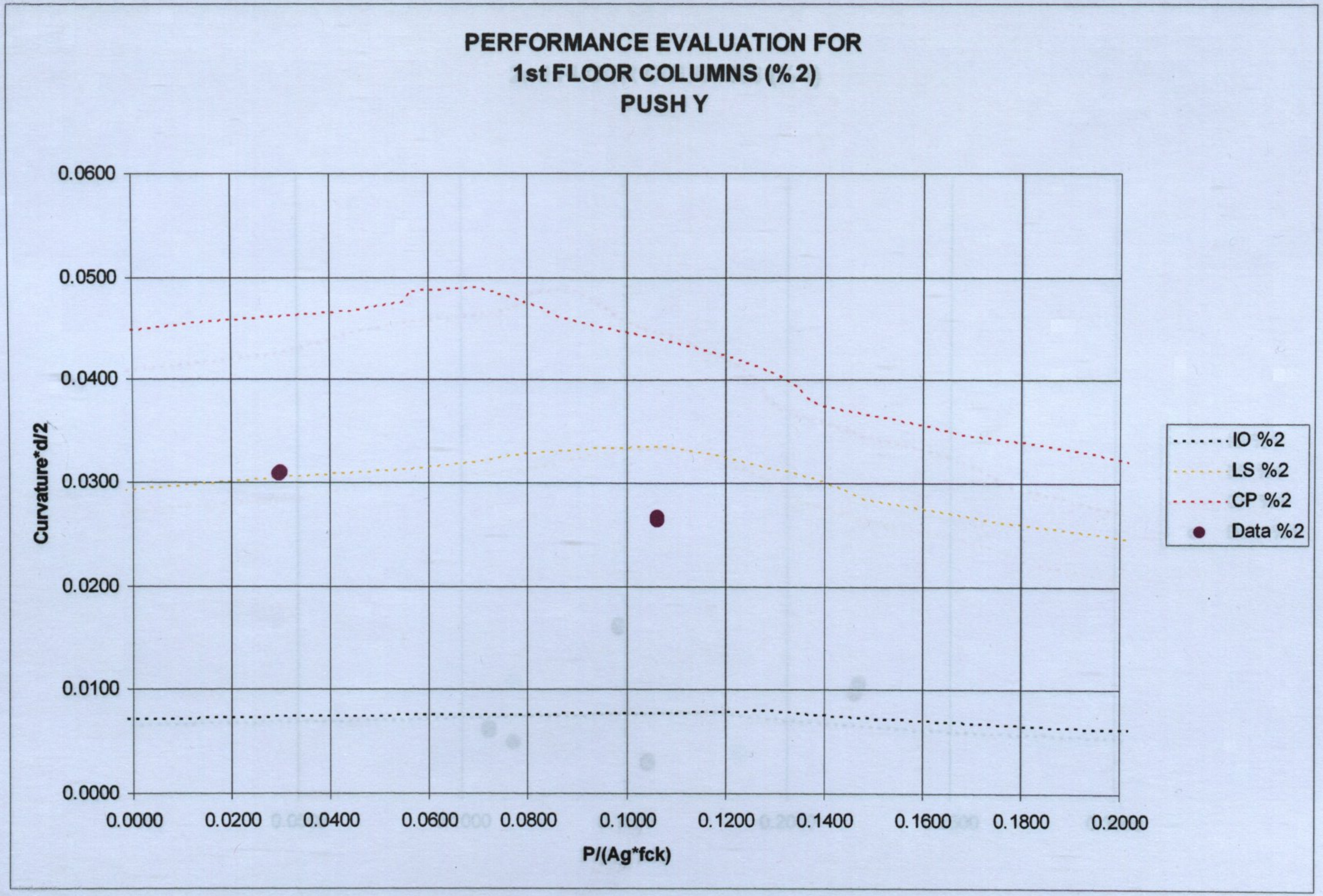
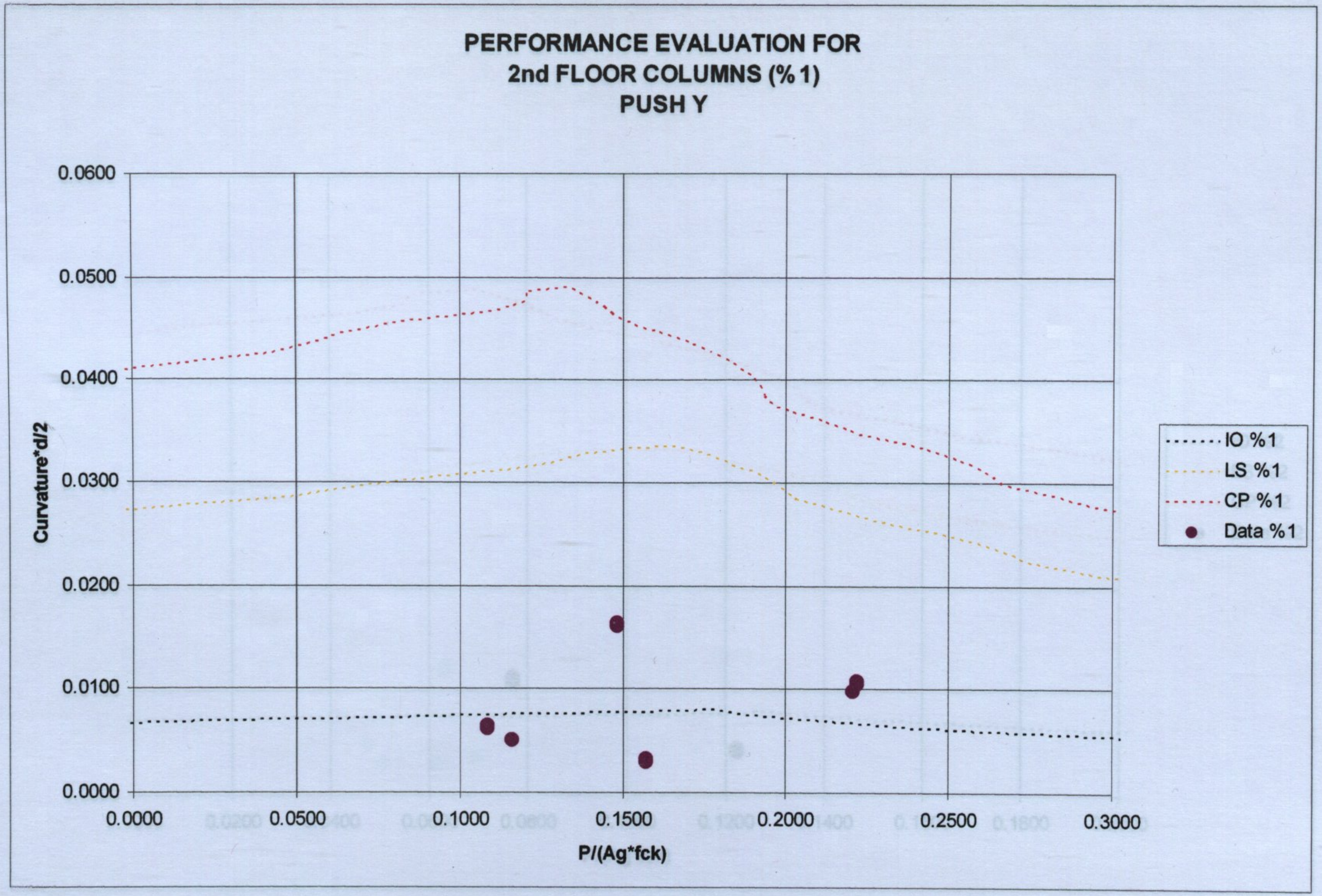
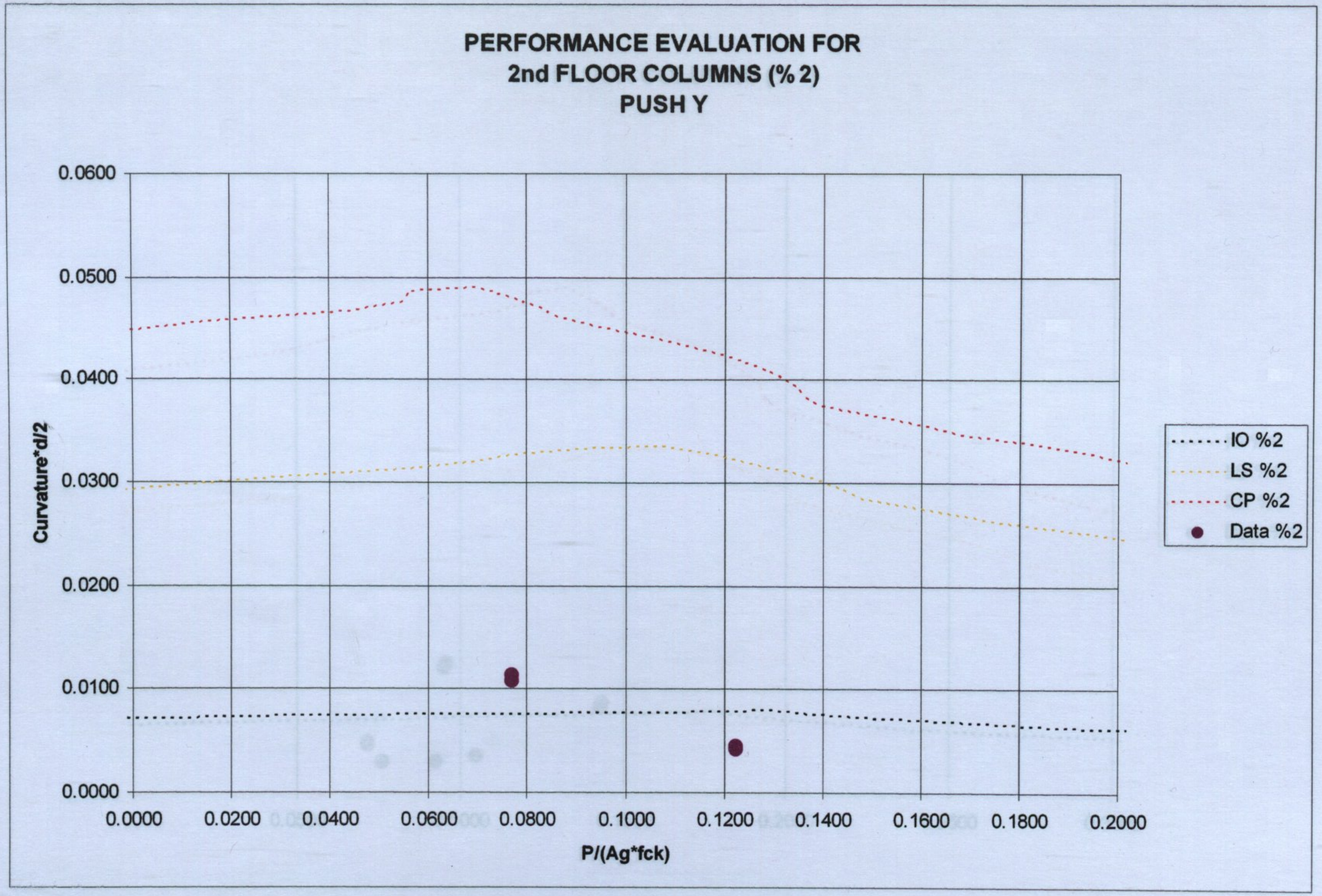


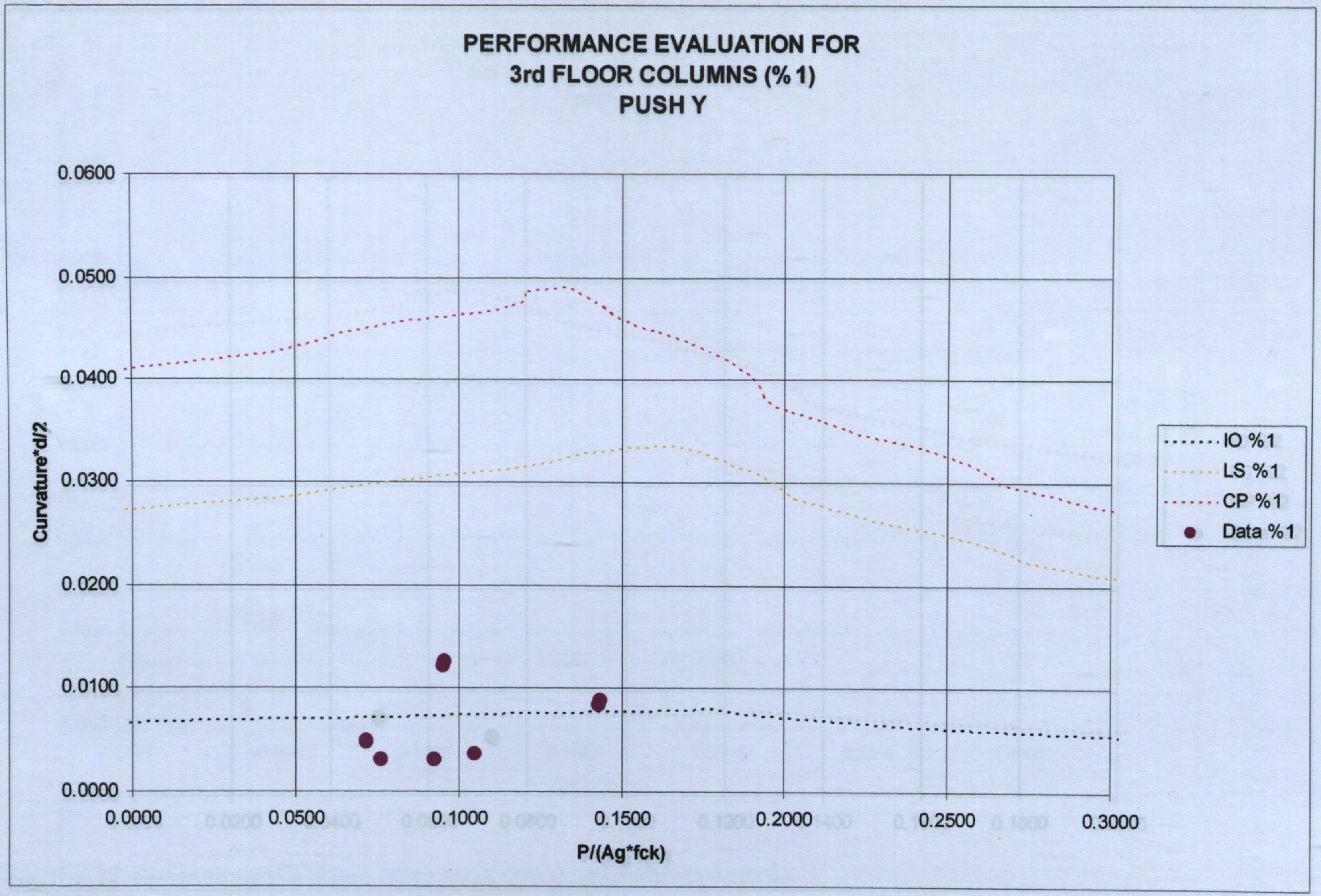
Figure 4.21. Hinge locations for X directional pushover analysis

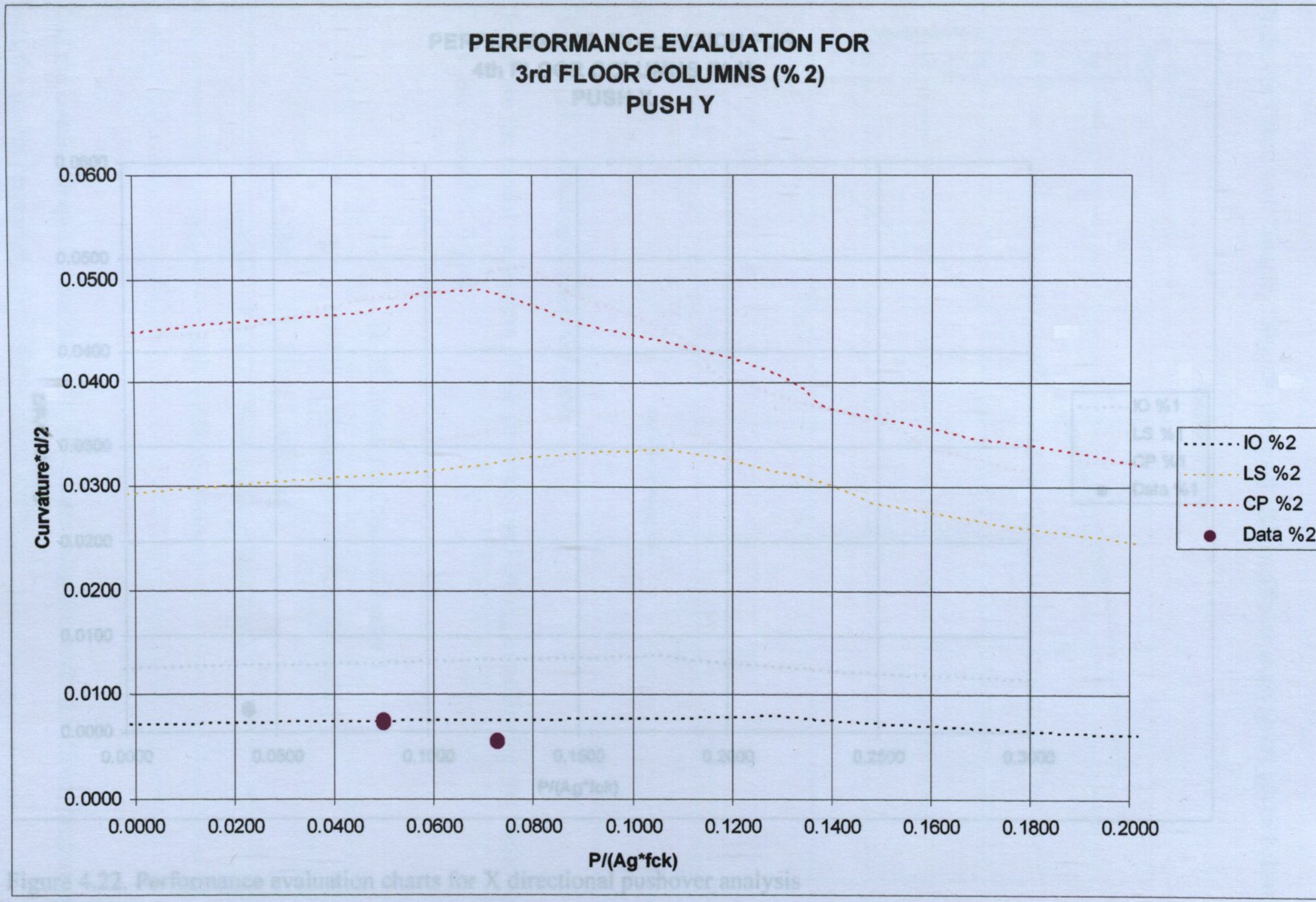

PERFORMANCE EVALUATION FOR
1st FLOOR COLUMNS (%1)
PUSH Y
Curvature*d/2
0.0600
0.0500
0.0400
0.0300
0.0200
0.0100
0.0000
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
P/(Ag*fck)
IO %1
LS %1
CP %1
Data %1


PERFORMANCE EVALUATION FOR
1st FLOOR COLUMNS (%2)
PUSH Y
Curvature*d/2
0.0600
0.0500
0.0400
0.0300
0.0200
0.0100
0.0000
0.0000
0.0200
0.0400
0.0600
0.0800
0.1000
0.1200
0.1400
0.1600
0.1800
0.2000
P/(Ag*fck)
IO %2
LS %2
CP %2
Data %2


PERFORMANCE EVALUATION FOR
2nd FLOOR COLUMNS (%1)
PUSH Y
Curvature*d/2
0.0600
0.0500
0.0400
0.0300
0.0200
0.0100
0.0000
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
P/(Ag*fck)
IO %1
LS %1
CP %1
Data %1


PERFORMANCE EVALUATION FOR
2nd FLOOR COLUMNS (% 2)
PUSH Y
Curvature*d/2
0.0600
0.0500
0.0400
0.0300
0.0200
0.0100
0.0000
0.0000
0.0200
0.0400
0.0600
0.0800
0.1000
0.1200
0.1400
0.1600
0.1800
0.2000
P/(Ag*fck)
IO %2
LS %2
CP %2
Data %2


PERFORMANCE EVALUATION FOR
3rd FLOOR COLUMNS (%1)
PUSH Y
Curvature*d/2
0.0600
0.0500
0.0400
0.0300
0.0200
0.0100
0.0000
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
P/(Ag*fck)
IO %1
LS %1
CP %1
Data %1


PERFORMANCE EVALUATION FOR
3rd FLOOR COLUMNS (% 2)
PUSH Y
Curvature*d/2
0.0600
0.0500
0.0400
0.0300
0.0200
0.0100
0.0000
0.0000
0.0200
0.0400
0.0600
0.0800
0.1000
0.1200
0.1400
0.1600
0.1800
0.2000
P/(Ag*fck)
IO %2
LS %2
CP %2
Data %2

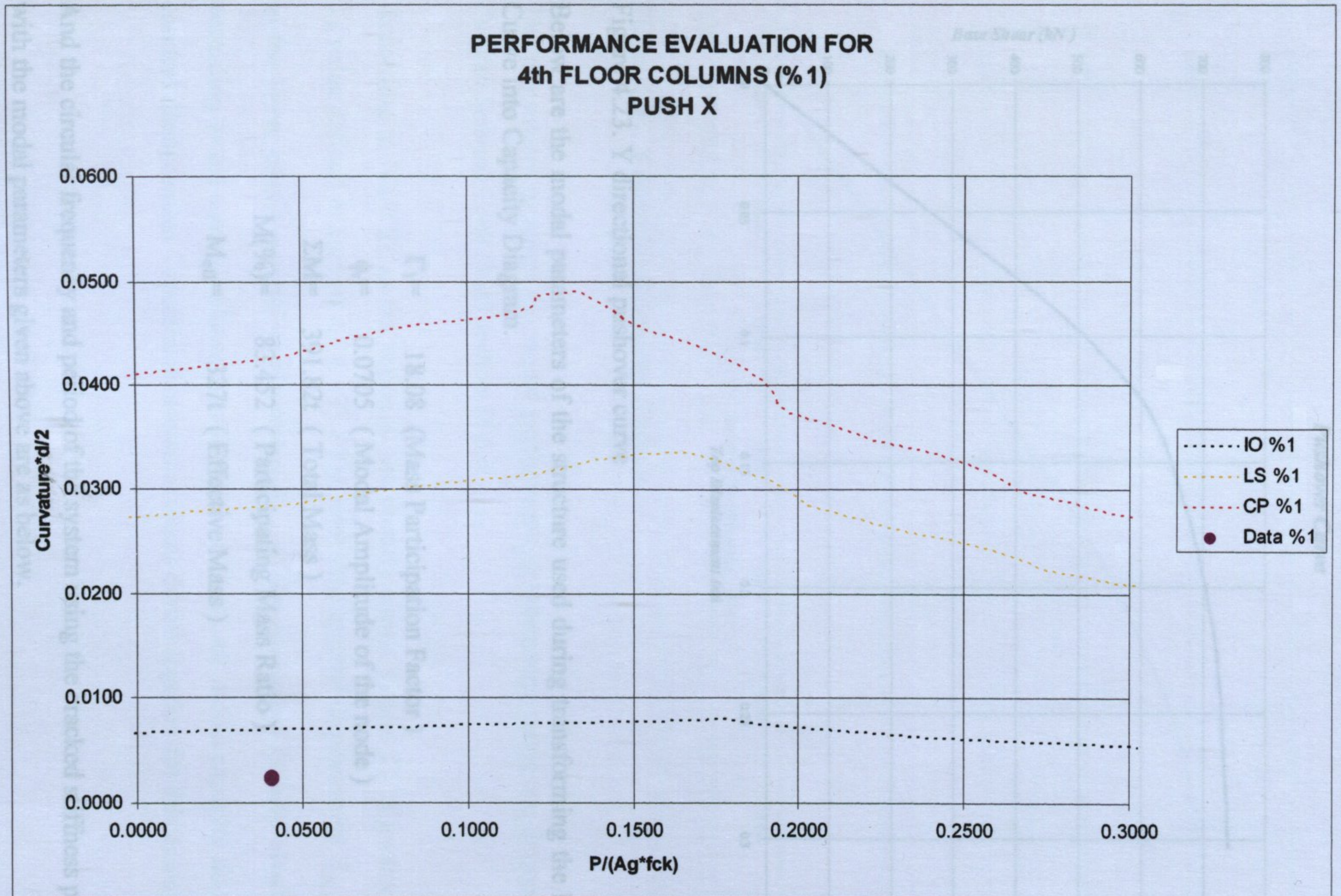


Figure 4.22. Performance evaluation charts for X directional pushover analysis

Pushover Analysis Results for the Direction Y

Pushover Curve and Capacity Diagram obtained after nonlinear analysis are given below.

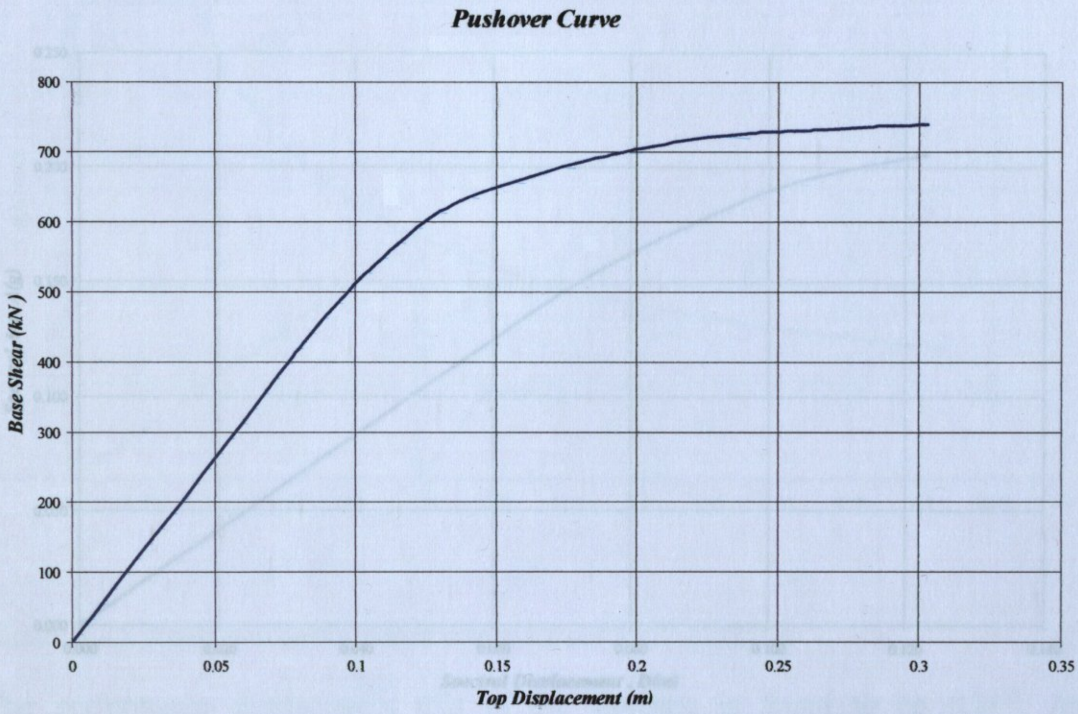


Figure 4.23. Y directional pushover curve

Below are the modal parameters of the structure used during transforming the Pushover Curve into Capacity Diagram.

$\Gamma_1=$ 18.08 (Mass Participation Factor)

$\phi_r=$ 0.0705 (Modal Amplitude of the node)

$\Sigma M=$ 391.82t (Total Mass)

M(%)= 83.452 (Participating Mass Ratio)

$M_{eff}=$ 327t (Effective Mass)

And the circular frequency and period of the system using the cracked stiffness properties with the modal parameters given above are as below.

$\omega^2 =$ 20.34 rad/sec

T= 1.393 sec

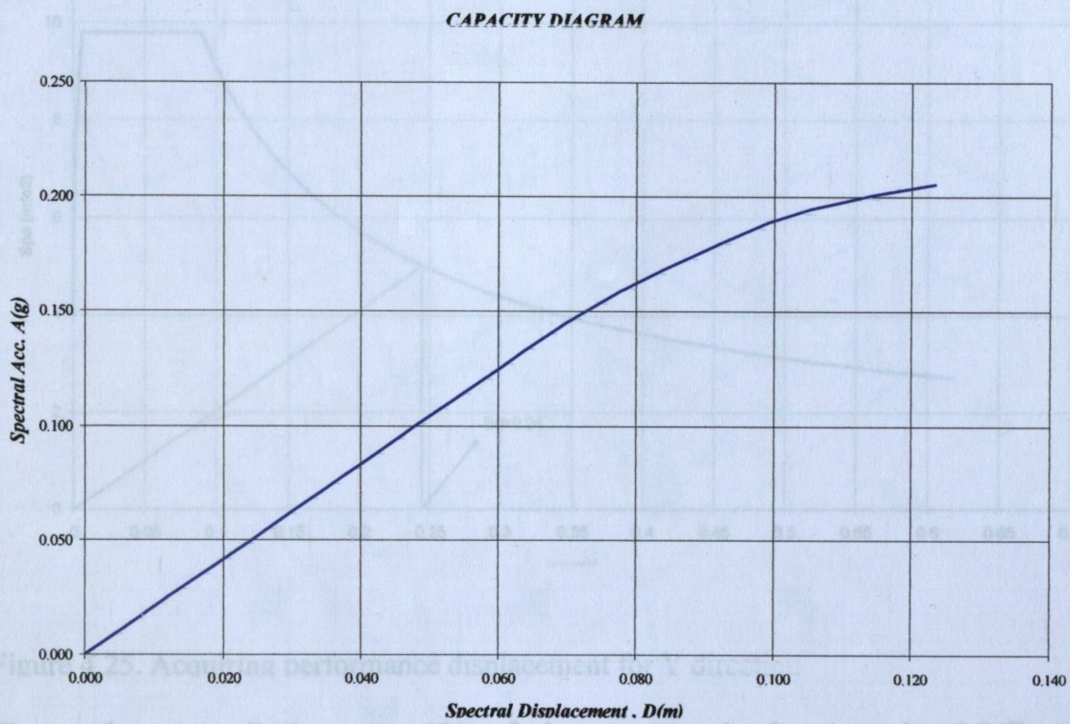


Figure 4.24. Y directional capacity diagram

The procedures to attain the modal performance displacement value are defined at *New Turkish Seismic Code Draft section 13.6.8*

According to NTSCD, as the period of the linearly elastic system T_1, is greater than the T_B value defined *in Section 4.1.4.* $(T_1 \geq T_B)$, nonlinear spectral displacement $S_{di,1}$ is equal to the linear elastic spectral displacement $S_{de,1}$ of the equivalent linearly elastic system according to the equal displacement rule. Below charts are modal capacity diagram and spectral displacement – spectral acceleration charts drawn together for this example.

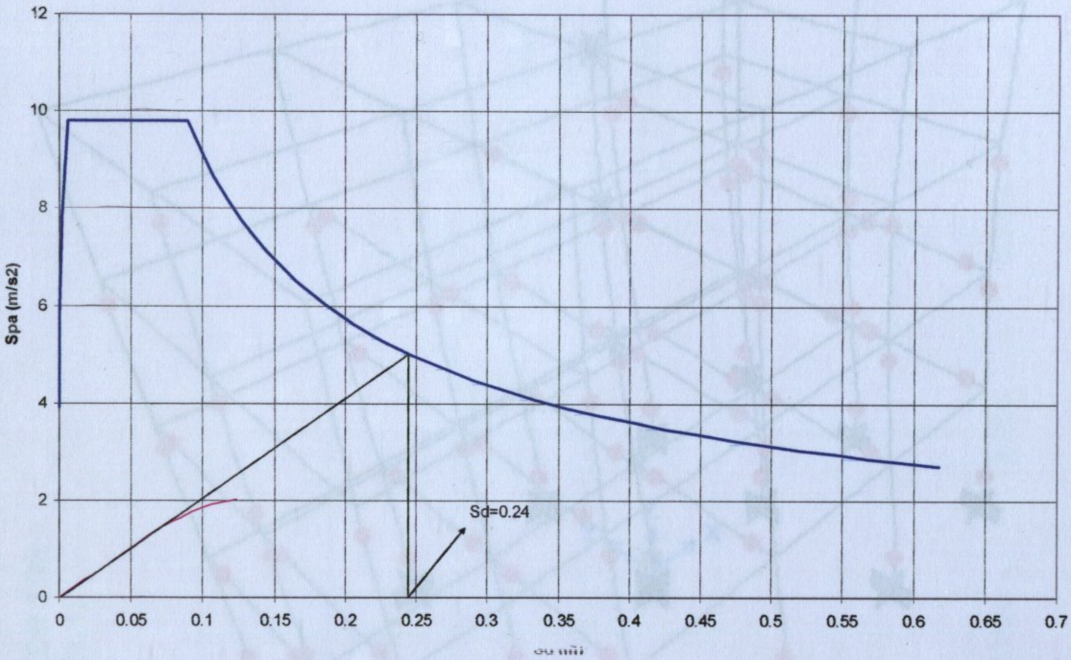


Figure 4.25. Acquiring performance displacement for Y direction

The performance displacement (S_d) of the structure is found to be 0.24^m. And performance evaluation for the structure is done by considering the plastic hinge rotations developed after the pushover analysis for this performance displacement.

Performance evaluation charts for different reinforcement ratios and hinge locations are given below.

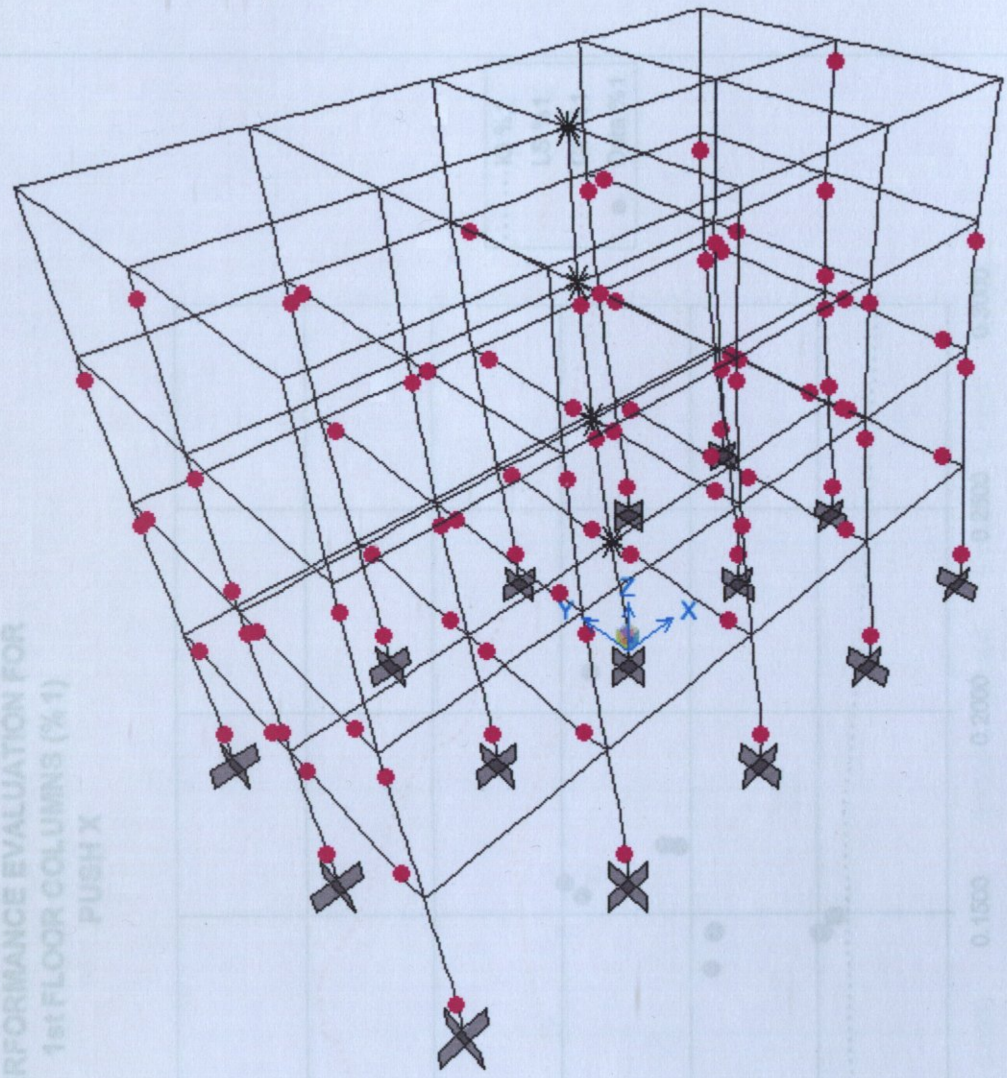
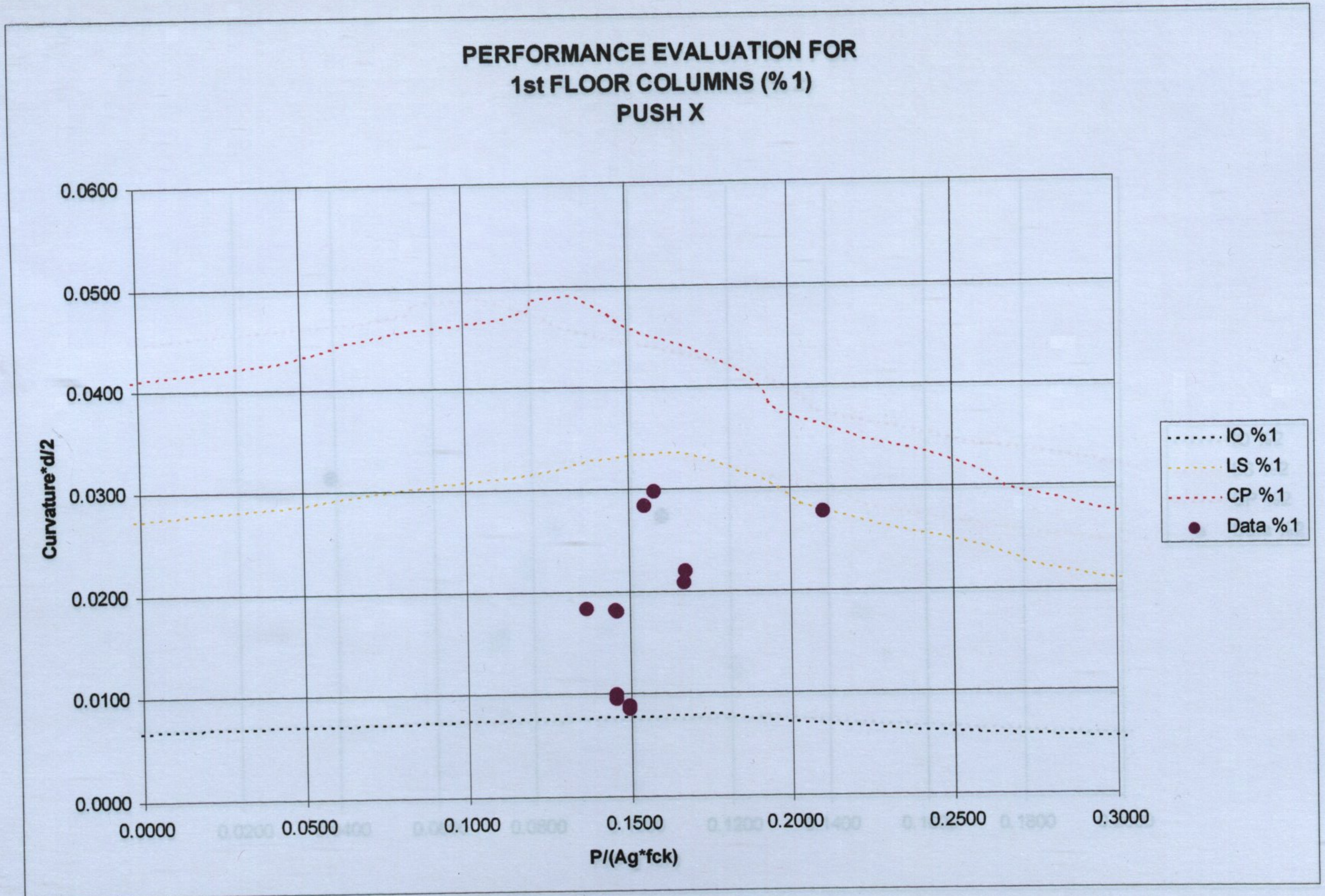
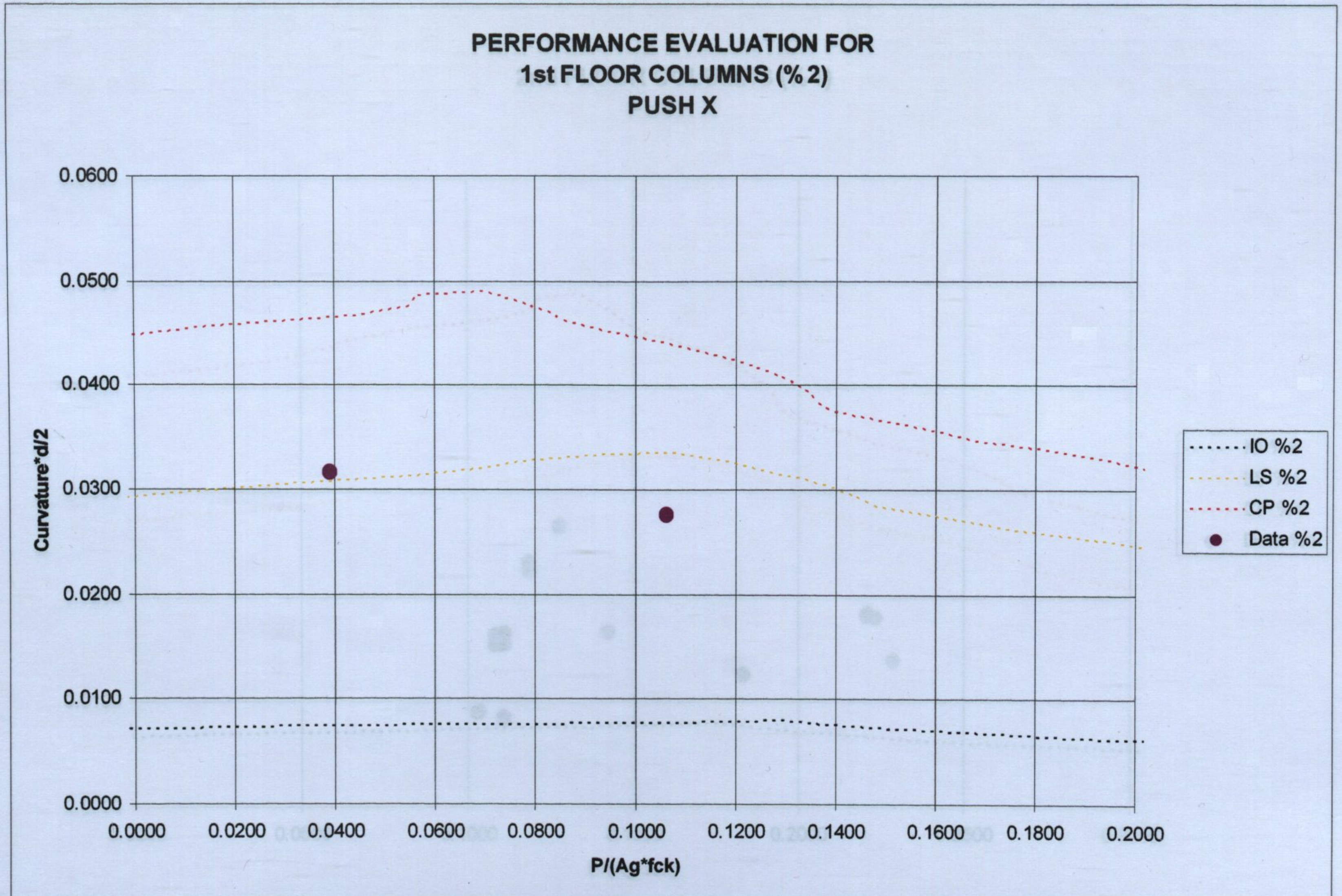
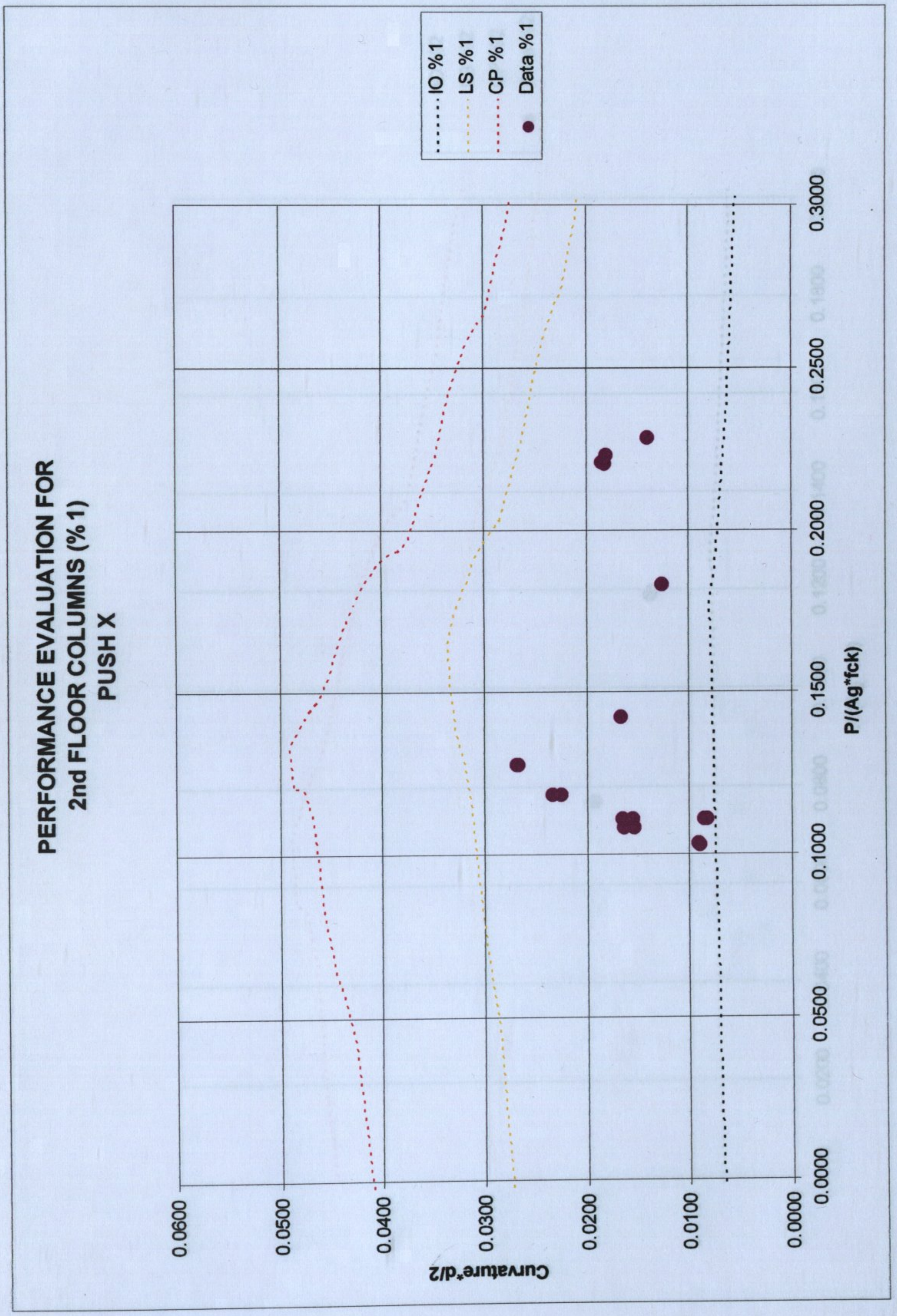
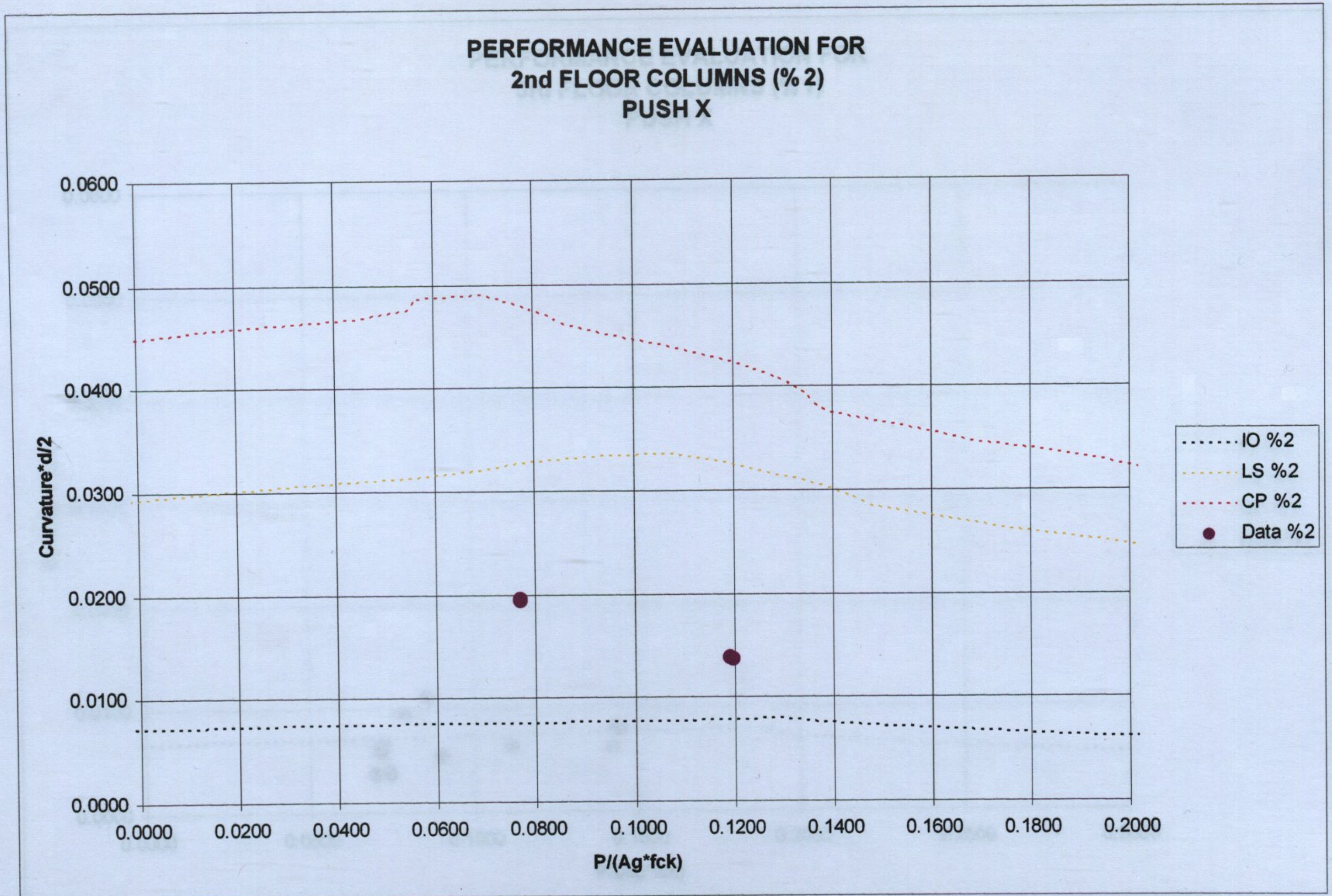


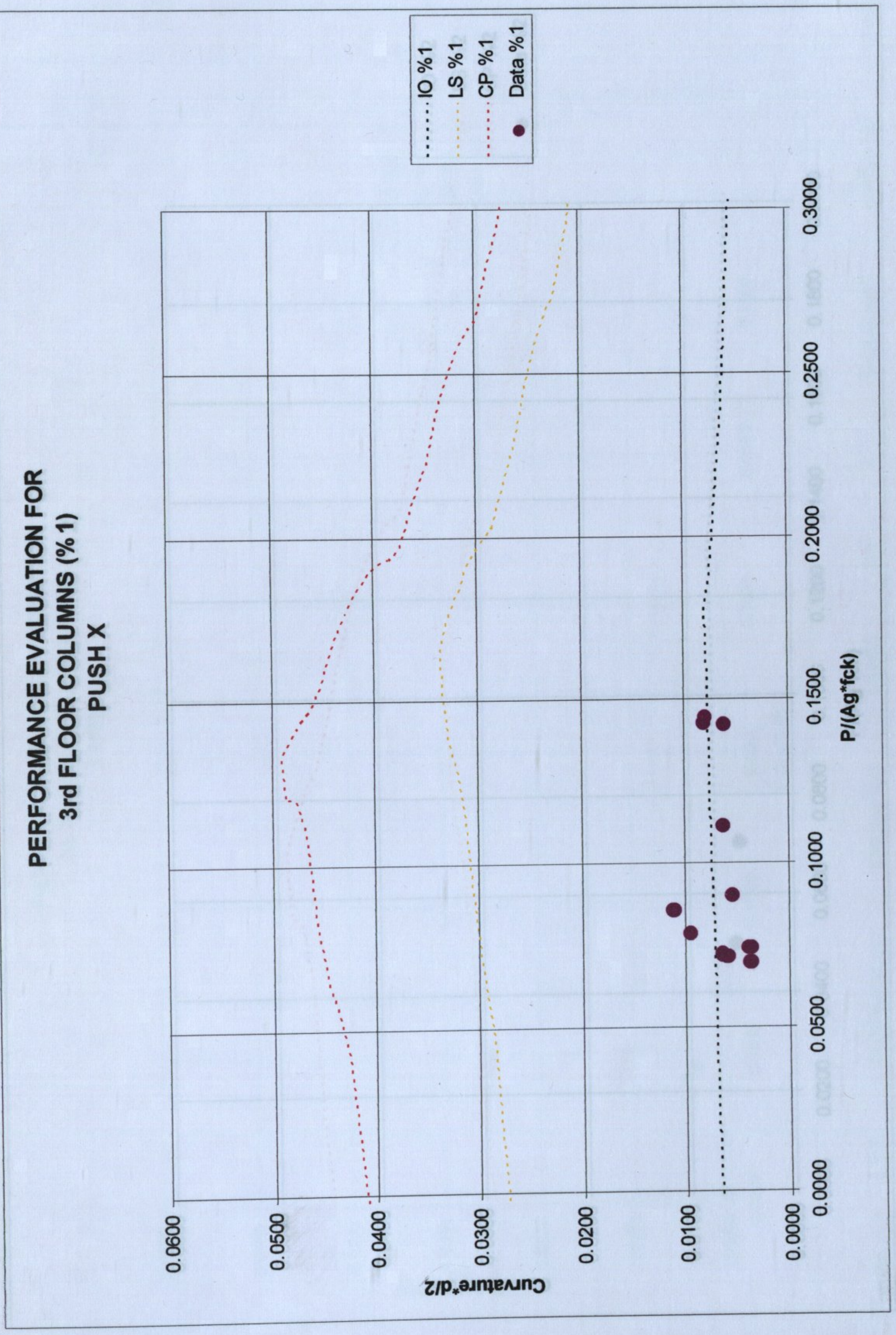
Figure 4.26. Hinge locations for Y directional pushover analysis


PERFORMANCE EVALUATION FOR
1st FLOOR COLUMNS (%1)
PUSH X
0.0600
0.0500
0.0400
0.0300
0.0200
0.0100
0.0000
Curvature*d/2
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
P/(Ag*fck)
IO %1
LS %1
CP %1
Data %1

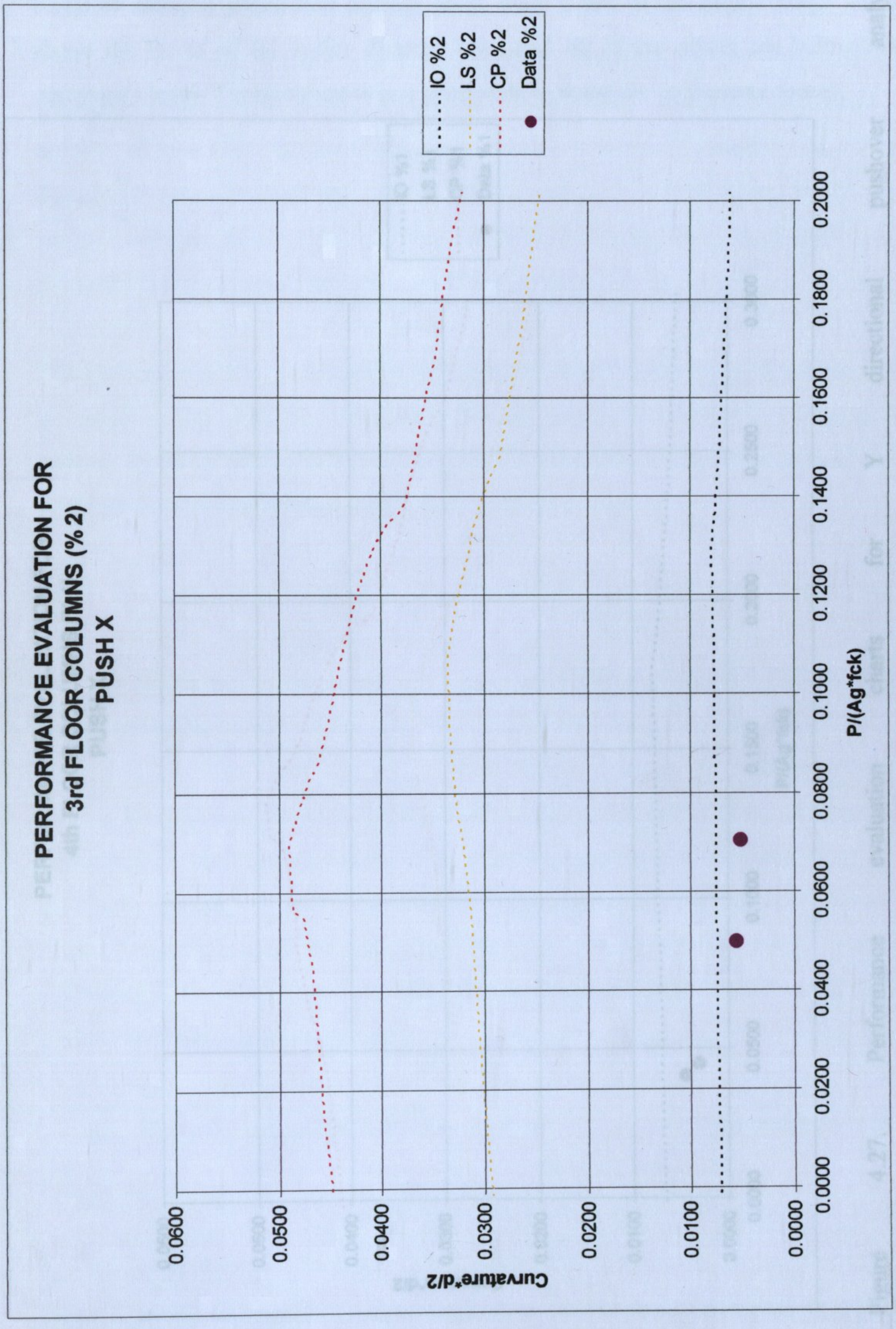

PERFORMANCE EVALUATION FOR
1st FLOOR COLUMNS (% 2)
PUSH X
Curvature*d/2
0.0600
0.0500
0.0400
0.0300
0.0200
0.0100
0.0000
0.0000
0.0200
0.0400
0.0600
0.0800
0.1000
0.1200
0.1400
0.1600
0.1800
0.2000
P/(Ag*fck)
IO %2
LS %2
CP %2
Data %2


PERFORMANCE EVALUATION FOR
2nd FLOOR COLUMNS (%1)
PUSH X
Curvature*d/2
0.0600
0.0500
0.0400
0.0300
0.0200
0.0100
0.0000
P/(Ag*fck)
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
IO %1
LS %1
CP %1
Data %1


PERFORMANCE EVALUATION FOR
2nd FLOOR COLUMNS (% 2)
PUSH X
Curvature*d/2
0.0600
0.0500
0.0400
0.0300
0.0200
0.0100
0.0000
0.0000
0.0200
0.0400
0.0600
0.0800
0.1000
0.1200
0.1400
0.1600
0.1800
0.2000
P/(Ag*fck)
IO %2
LS %2
CP %2
Data %2



PERFORMANCE EVALUATION FOR
3rd FLOOR COLUMNS (%1)
PUSH X
IO %1
LS %1
CP %1
Data %1
Curvature*d/2
0.0600
0.0500
0.0400
0.0300
0.0200
0.0100
0.0000
P/(Ag*fck)
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000



PERFORMANCE EVALUATION FOR
3rd FLOOR COLUMNS (%2)
PUSH X
Curvature*d/2
P/(Ag*fck)
0.0000
0.0100
0.0200
0.0300
0.0400
0.0500
0.0600
0.0000
0.0200
0.0400
0.0600
0.0800
0.1000
0.1200
0.1400
0.1600
0.1800
0.2000
IO %2
LS %2
CP %2
Data %2

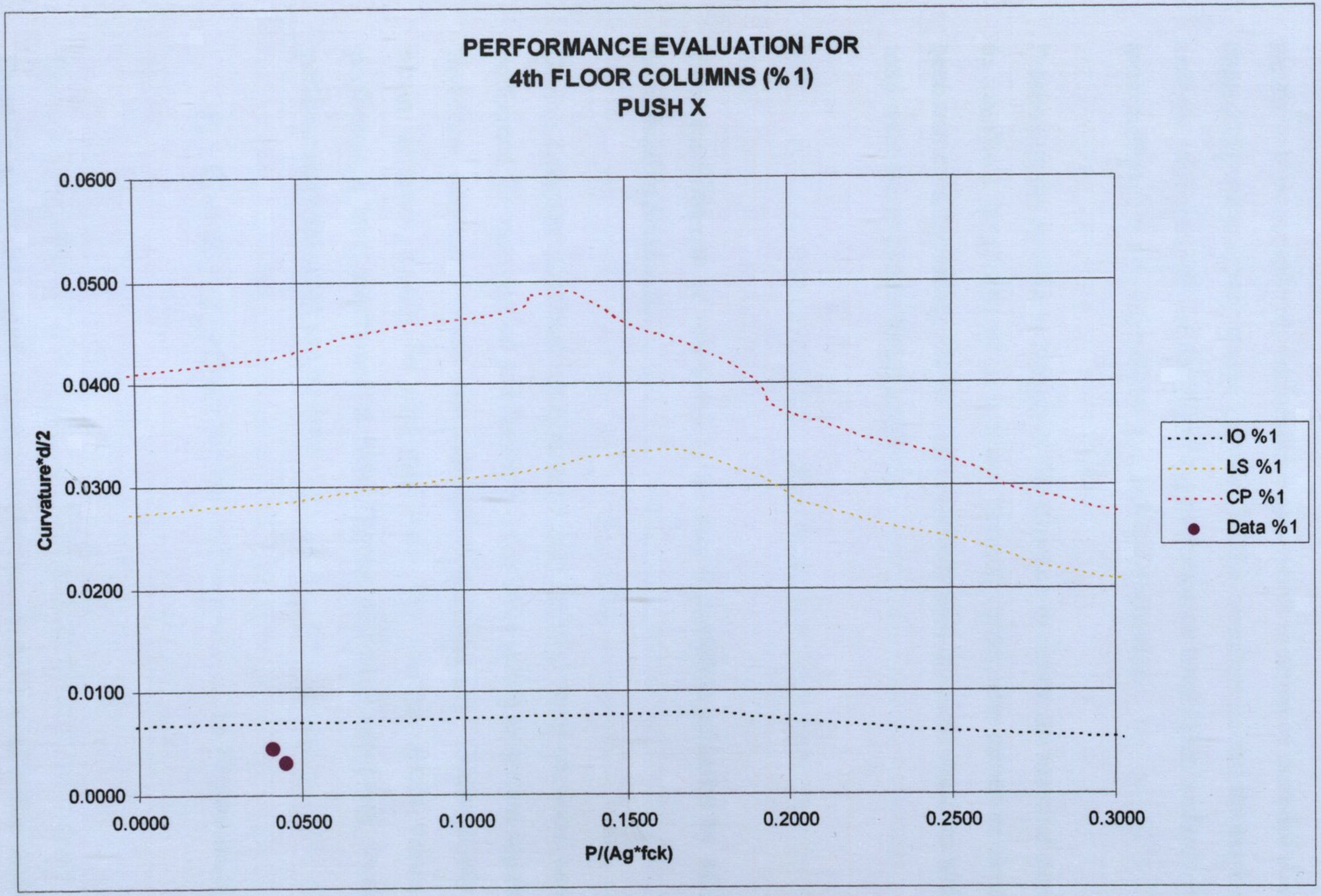


Figure 4.27. Performance evaluation charts for Y directional pushover analysis

None of the plastic hinges occurred in the structure for both directions do not exceed the limits of *collapse prevention* damage level. Only a few of the plastic hinge rotations are above the limits of *life safety* damage level and all of the others are below *immediate occupancy* level. The structure is in a good state to resist the earthquake forces.

5. CONCLUSION

With *New Turkish Seismic Code,* in Nonlinear analysis of the reinforced concrete structures under seismic attack, a different performance evaluation criteria using material strains will be considered. Performance evaluation in terms of material strains is done by comparing the material strains occurred in the members of the structure after nonlinear analysis with the limits of the regarding performance level. This method is thought to give more realistic results with respect to rotational evaluation.

Proceeding the nonlinear analysis of the structure to obtain the material strains, first step is to transform the plastic hinge rotation demand values into curvature demands. This has been achieved by adding plastic hinge rotations with the yield rotations and dividing these total rotations to plastic hinge length L_p.

$$L_p = 0.5\,d \tag{5.1}$$

Yield rotations can be calculated by an easy formulation achieved by *M.J.N Priestley* as mentioned in *Section 3.*

During a regular nonlinear analysis procedure, several cross sectional analysis should be performed for each section and for every axial force acting to section separately in order to transform curvature values into material strain demands. Consequently these material strain demands should be compared with the limiting strain values for regarding performance level mentioned in *New Turkish Seismic Code Draft Section 13.6.10* and performance evaluation can be done.

$$\theta_y + \theta_p \rightarrow \theta_t \rightarrow \phi \rightarrow \textit{Cross Section Analyses} \rightarrow \varepsilon_s, \varepsilon_c \Rightarrow \textit{Performance Evaluation}$$

By utilizing the derived performance evaluation charts formed in this study, separate cross sectional analyses under different axial forces for transforming the rotations into strains are not necessary. Plotting the rotation-axial force ratio couples into the charts and check whether these values satisfy the regarding performance level will suffice.

$$\theta_y + \theta_p \rightarrow \theta_t \Rightarrow Performance\ Evaluation$$

In order to check which parameters effect the performance evaluation charts, several analysis have been performed and it has been concluded that performance evaluation charts are independent to the section dimensions but dependent the concrete and steel strength, reinforcement ratio and confinement level.

Confinement level is the most effective factor on the performance evaluation charts. With an increase in confinement level the rotation capacity increases also in a direct proportion, where unconfined section results give the poorest deformation capacities.

Effect of reinforcement ratio and concrete strength varies with the change in axial force ratio.

1. If more than half of the section area is opposed to compression forces and concrete strains reach the limits of the regarding performance level, rotation capacity decreases with the increase in reinforcement ratio and concrete strength.

$$A_S, f_{ck} \uparrow \theta \downarrow$$

2. If more than half of the section area is opposed to tension forces and concrete strains reach the limits of the regarding performance level, rotation capacity increases with the increase in reinforcement ratio.

$$A_S, f_{ck} \uparrow \theta \uparrow$$

Steels having lower tensile capacity as in S220 (f_{yk}=220MPa) grade steels, are more ductile than the steels having higher tensile capacity as in S420 (f_{yk}=420MPa) grade steels. This effects the rotation capacity of the section and rotation capacity decreases with the increase in steel strength.

$f_{yk} \uparrow \theta \downarrow$

These *Strain Based Performance Evaluation Charts* are intended to be very useful for the structural engineers who will practice nonlinear analysis using *New Turkish Seismic Code* in the future. However it should be noticed that these charts are formed only for rectangular reinforced columns with limited concrete-steel grade combinations. Therefore further studies should be performed for other combinations and beams.

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