

AN EVALUATION OF THE BEHAVIOUR OF RC BRIDGE PIERS UNDER
VERTICAL AND HORIZONTAL COMPONENTS OF EARTHQUAKE GROUND
MOTION

by

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ABSTRACT

AN EVALUATION OF THE BEHAVIOUR OF RC BRIDGE PIERS UNDER VERTICAL AND HORIZONTAL COMPONENTS OF EARTHQUAKE GROUND MOTION

Reinforced concrete (RC) bridges with single piers have been used extensively in Turkish highways. During the past earthquakes in Japan, USA and New Zealand extensive damages occurred in the single piers indicates the inadequacy of seismic design. In the design of the concrete bridge piers, generally horizontal earthquake forces are taken into consideration. An additional P- Δ effect occurs when the horizontal earthquake effect and the vertical earthquake effect act together on the bridge piers with a large beam cantilever width. The effect of additional P- Δ effect under the effect of vertical earthquake may increase the bending moment and shear forces values in the columns and the piers supporting the bridge may experience damage.

Another circumstance encountered in large-scale projects is the use of filler material as a dump / storage area under bridge piers. This causes a certain part of the bridge pier heights to remain under the soil filling material and shortens the free pier height. For a single column pier of a conventional bridge maximum moment and shear values occur at the bottom of the column under seismic forces. This region where the maximum stress occurs is designed to be the plastic hinge region of the column. Maximum shear and longitudinal reinforcement used in the plastic hinge region to provide the conditions is stated in the regulations. When the part of the column is buried under the soil, maximum moment and shear force values of the bridge columns may not occur at the bottom of the column but at the upper end of the buried height. Therefore, the critical section of the column will be above the bottom of the column which was not considered in the seismic design. The amount of longitudinal and shear reinforcement may also be inadequate than the required reinforcement to resist the seismic forces which may cause the flexural and shear failure.

In this study, a single 30 m-tall RC pier of a conventional bridge is examined under horizontal and vertical components of three separate earthquake records for three different deck to pier flexural stiffness ratios. Conventional bridges are constructed with movement joints and connections. The pier has a monolithic 15m-wide cap beam. The height of the cap beam is increased to provide different flexural stiffness ratios between column and cap beam. As the flexural stiffness of the cap beam is increased, the change in the column shear force and moment due to additional P- Δ effect of vertical component of earthquake is investigated. As a result, only the horizontal impact of the earthquake and the effect of both horizontal and vertical earthquakes are compared for the column forces.

Secondly, the single pier of the conventional bridge with the same geometric properties is investigated for the cases that it is buried under 5 m, 10 m and 15 m of earth fill. The horizontal components of the same earthquake records are used in the analysis. Force-deformation relation of the soil is represented by non-linear p-y springs for the buried cases.

ÖZET

BETONARME KÖPRÜ AYAKLARININ DEPREM KAYITLARININ YATAY VE DÜŞEY BİLEŞENLERİ ALTINDAKİ DAVRANIŞLARININ İRDELENMESİ

Türkiye'deki karayollarında tek kolonlu betonarme köprüler oldukça yaygındır. Geçmişte Japonya, Amerika ve Yeni Zelanda'da meydana gelen depremlerde tek kolonlu köprü ayaklarında ciddi hasarlar meydana gelmesi sismik dizaynın yetersiz olduğunu göstermektedir. Betonarme ayaklı köprülerin projelendirilmesinde genellikle yatay yöndeki deprem kuvvetleri dikkate alınmaktadır. Özellikle başlık kirişi konsol genişliği fazla olan köprü ayaklarında yatay deprem etkisi ile düşey deprem etkisinin birlikte etki etmesi durumunda ilave P- Δ etkisi oluşmaktadır. Düşey deprem etkisi altında oluşan ilave P- Δ etkisi kolonlardaki eğilme momenti ve kesme kuvvetleri değerlerini arttırarak köprü ayaklarında hasar oluşmasına sebep olabilmektedir.

Özellikle büyük ölçekli projelerde karşılaşılan bir diğer durum ise, köprü ayakları civarının dolgu malzemesi döküm/depolama sahası olarak kullanılmasıdır. Bu durum köprü ayak yüksekliklerinin belli bir kısmının toprak dolgu malzemesi altında kalarak serbest ayak yüksekliğinin kısılmasına sebep olmaktadır. Tek kolonlu konvansiyonel bir köprü ayağında deprem yükleri altındaki maksimum eğilme momenti ve kesme kuvveti değerleri kolon alt ucunda oluşmaktadır. Maksimum kuvvetlerin olduğu bu bölge plastik mafsallık bölgesi olarak tasarlanmaktadır. Tasarımda bu bölge dikkate alınarak gerekli kesme ve ana donatı belirlenmektedir. Fakat kolonun bir kısmının toprağa gömülmesi durumunda maksimum kuvvetler kolon alt ucunda değil gömülü yüksekliğin üst ucunda meydana gelebilecektir. Daha üst bölgelerde ilk tasarımda kesme ve ana donatı sayısı ve aralığında azaltma yapıldı ise kesme ve eğilme açısından kolon kesitinin yetersiz kalması durumu ortaya çıkabilecektir.

Bu çalışmada, 30 m yüksekliğindeki 15 m başlık kirişi genişliğine sahip tek kolonlu bir köprü ayağı 3 ayrı deprem kaydının düşey ve yatay bileşeni altında 3 farklı başlık kirişi/kolon eğilme rijitliği oranı için incelenmiştir. Kolon ve başlık kirişi arasındaki 3 farklı

eğilme rijitliği oranını elde edebilmek için başlık kirişi yükseklikleri arttırılmıştır. Başlık kirişi eğilme rijitliği arttıkça düşey deprem etkisi ile oluşacak ilave $P-\Delta$ etkisi sebebiyle kolonda meydana gelen kesme kuvveti ve eğilme momenti değişimi incelenmiştir. Depremi sadece yatay bileşenin etki etmesi durumu ve hem yatay hem düşey depremin birlikte etki etmesi durumu karşılaştırılmıştır.

İkinci olarak aynı geometrik özelliklere sahip köprü ayağının 5m-10m ve 15 m toprak dolgu ile gömülü olduğu durum incelenmiştir. Analizler için aynı deprem kayıtlarının yatay bileşenleri kullanılmıştır. Yatay yöndeki toprak kuvvet- deformasyon ilişkisi doğrusal olmayan p-y yayları ile temsil edilerek meydana gelen kesit tesirleri kolonun gömülü olmadığı durum ile kıyaslanmıştır

TABLE OF CONTENTS

ACKNOWLEDGEMENTS	iii
ABSTRACT	iv
ÖZET	vi
1. INTRODUCTION	1
1.1. Objectives of the Study	1
1.2. Literature Review	2
2. DESCRIPTION OF SINGLE COLUMN BRIDGE PIER	4
2.1. Material Properties of Pier	7
2.1.1. Concrete Properties.....	7
3. SELECTION OF GROUND MOTION RECORDS	8
3.1. Horizontal Acceleration, Velocity, Displacement Time Histories Kobe (Jan. 16, 1995)- Station: Port Island	9
3.2. Vertical Acceleration, Velocity, Displacement Time Histories Kobe (Jan. 16, 1995)- Station: Port Island	10
3.3. Horizontal Acceleration, Velocity, Displacement Time Histories Kocaeli (Aug. 17, 1999) - Station: Sakarya.....	11
3.4. Vertical Acceleration, Velocity, Displacement Time Histories Kocaeli (Aug. 17, 1999) - Station: Sakarya.....	12
3.5. Horizontal Acceleration, Velocity, Displacement Time Histories Kocaeli (Aug. 17, 1999)- Station: Yarımca150	13
3.6. Vertical Acceleration, Velocity, Displacement Time Histories Kocaeli (Aug. 17, 1999)- Station: Yarımca150	14
4. STRUCTURAL MODELLING	15
4.1. Structural Model for Investigating the Effects of Vertical Component of Earthquake	15
4.1.1. Soil Stiffness Idealization	18
4.1.2. Loads Considered for Pier.....	21
4.1.3. Lumped Mass Analysis Model.....	24
4.1.4. Distributed Mass Analysis Model	27
5. ANALYSIS RESULTS	28

5.1. Modal Analysis results.....	28
5.1.1. Modal Shapes and Periods for Lumped Mass Analysis Model	29
5.1.2. Modal Shapes and Periods for Distributed Mass Analysis Model.....	32
5.2. Spectral Accelerations for Vertical and Horizontal Component of Ground Motions Records.....	35
5.3. Time History Analysis Results under Horizontal-only Ground Motion and Both Horizontal and Vertical Ground Motion.....	37
5.3.1. Column Shear Force Diagrams for Flexural Stiffness Ratio: 0.30.....	38
5.3.2. Column Moment Diagrams for Flexural Stiffness Ratio: 0.30.....	40
5.3.3. Column Shear Force Diagrams for Flexural Stiffness Ratio: 0.60.....	42
5.3.4. Column Moment Diagrams for Flexural Stiffness Ratio: 0.60.....	44
5.3.5. Column Shear Force Diagrams for Flexural Stiffness Ratio: 0.90.....	46
5.3.6. Column Moment Diagrams for Flexural Stiffness Ratio: 0.90.....	48
6. SEISMIC PERFORMANCE OF SINGLE COLUMN BRIDGE PIER UNDER DIFFERENT DEPTH OF SOIL EMBEDMENT	50
6.1. P-y Curves for Lateral Soil Resistance-Deflection.....	50
6.2. Mode Shapes and Periods for 15 m Cap Beam Width	60
6.2.1. Mode Shape and Period for No Soil Embedment Case	60
6.2.2. Mode Shape and Period for 5 m Soil Embedment Case.....	61
6.2.3. Mode shape and period for 10 m Soil Embedment case.....	62
6.2.4. Mode Shape and Period for 15 m Soil Embedment	63
6.3. Spectral Accelerations for Modal Periods in case of Different Soil embedment Depths	64
6.4. Shear force Diagrams for Different Soil Embedment Depths	66
6.5. Moment Diagrams for Different Soil Embedment Depths	68
7. CONCLUSIONS	70
REFERENCES.....	72

LIST OF FIGURES

Figure 2.1. Section drawings of the pier and the cap beam.....	5
Figure 2.2. Superstructure cross section.....	6
Figure 2.3. 3D view of the pier and the cap beam	6
Figure 2.4. Stress-strain diagram for concrete.....	7
Figure 3.1. Horizontal acceleration time history Kobe (Jan. 16, 1995) - station: Port Island	9
Figure 3.2. Horizontal velocity time history Kobe (Jan. 16, 1995) - station: Port Island	9
Figure 3.3. Horizontal displacement time history Kobe (Jan. 16, 1995) - station: Port Island	9
Figure 3.4. Vertical acceleration time history Kobe (Jan. 16, 1995) - station: Port Island .	10
Figure 3.5. Vertical velocity time history Kobe (Jan. 16, 1995) - station: Port Island	10
Figure 3.6. Vertical displacement time history Kobe (Jan. 16, 1995) - station: Port Island	10
Figure 3.7. Horizontal acceleration time history Kocaeli (Aug. 17, 1999)- station: Sakarya	11
Figure 3.8. Horizontal velocity time history Kocaeli (Aug. 17, 1999)- station: Sakarya ...	11
Figure 3.9. Horizontal displacement time history Kocaeli (Aug. 17,1999)-station: Sakarya	11
Figure 3.10. Vertical acceleration time history Kocaeli (Aug. 17, 1999)- station: Sakarya	12
Figure 3.11. Vertical velocity time history Kocaeli (Aug. 17, 1999)- station: Sakarya.....	12

Figure 3.12. Vertical displacement time history Kocaeli (Aug. 17, 1999)- station: Sakarya	12
Figure 3.13. Horizontal acceleration time history Kocaeli (Aug.17, 1999)- station: Yarımca150.....	13
Figure 3.14. Horizontal velocity time history Kocaeli (Aug.17, 1999)-station: Yarımca150	13
Figure 3.15. Horizontal time history Kocaeli (Aug.17, 1999)-station Yarımca150	13
Figure 3.16. Vertical acceleration time history Kocaeli (Aug.17,1999)-station: Yarımca150	14
Figure 3.17. Vertical velocity time history Kocaeli (Aug.17, 1999)-station: Yarımca150.	14
Figure 3.18. Vertical time history Kocaeli (Aug.17, 1999)-station Yarımca150	14
Figure 4.1. Pier-cap beam flexural stiffness ratio: 0.30	16
Figure 4.2. Pier-cap beam flexural stiffness ratio: 0.60	16
Figure 4.3. Pier-cap beam flexural stiffness ratio: 0.90	16
Figure 4.4. Lumped mass analysis model.....	17
Figure 4.5. Distributed mass analysis model	17
Figure 4.6. Vertical and horizontal stiffness of the shallow foundation	18
Figure 4.7. Analysis model with linear elastic springs represents the vertical and horizontal stiffness of shallow footing foundation.	20
Figure 4.8. Superstructure cross section.....	21

Figure 4.9. Analysis model flexural stiffness ratio (cap beam / column) =0.30.....	24
Figure 4.10. Analysis model flexural stiffness ratio (cap beam / column) = 0.60.....	25
Figure 4.11. Analysis model flexural stiffness ratio (cap beam / column) = 0.90.....	26
Figure 4.12. Total deck load	27
Figure 5.1. Modal shape for flexural stiffness ratio (cap beam/column): 0.30.....	29
Figure 5.2. Modal shape for flexural stiffness ratio (cap beam/ column): 0.60.....	30
Figure 5.3. Modal shape for flexural stiffness ratio (cap beam/ column): 0.90.....	31
Figure 5.4. Modal shape for flexural stiffness ratio (cap beam/column): 0.30.....	32
Figure 5.6. Modal shape for flexural Stiffness Ratio (cap beam/column): 0.90.....	34
Figure 5.7. 5% damped elastic acceleration response spectra for Kobe earthquake vertical and horizontal components, the dots represent the spectral acceleration values for horizontal and vertical fundamental modal periods	35
Figure 5.8. 5% damped elastic acceleration response spectra for Sakarya station Kocaeli earthquake vertical and horizontal components, the dots represent the spectral acceleration values for horizontal and vertical fundamental modal periods	36
Figure 5.9. 5% damped elastic acceleration response spectra for Yarimca station Kocaeli earthquake vertical and horizontal components, the dots represent the spectral acceleration values for horizontal and vertical fundamental modal periods	36
Table 5.2. Shear forces (kN) under horizontal and both vertical and horizontal ground motion for flexural stiffness ratios: 0.30 – 0.60 – 0.90.....	37
Figure 5.10. Shear force diagrams: Kobe earthquake flexural stiffness ratio: 0.30.....	38

Figure 5.11. Shear force diagrams: Kocaeli earthquake Sakarya station flexural stiffness ratio: 0.30.....	39
Figure 5.12. Shear force diagrams: Kocaeli earthquake Yarimca station flexural stiffness ratio: 0.30.....	39
Figure 5.13. Moment diagrams: Kobe earthquake flexural stiffness ratio: 0.30	40
Figure 5.14. Moment diagrams: Kocaeli earthquake-Sakarya station flexural stiffness ratio: 0.30.....	40
Figure 5.15. Moment diagrams: Kocaeli earthquake-Yarimca station flexural stiffness ratio: 0.30.....	41
Figure 5.16. Shear force diagrams: Kobe earthquake flexural stiffness ratio: 0.60.....	42
Figure 5.17. Shear force diagrams: Kocaeli earthquake Sakarya station flexural stiffness ratio: 0.60.....	42
Figure 5.18. Shear force diagrams: Kocaeli earthquake Yarimca station flexural stiffness ratio: 0.60.....	43
Figure 5.19. Moment diagrams: Kobe earthquake flexural stiffness ratio: 0.60	44
Figure 5.20. Moment diagrams: Kocaeli earthquake-Sakarya station flexural stiffness ratio: 0.60.....	44
Figure 5.21. Moment diagrams: Kocaeli earthquake-Yarimca station flexural stiffness ratio: 0.60.....	45
Figure 5.22. Shear force diagrams: Kobe earthquake flexural stiffness ratio: 0.90.....	46
Figure 5.23. Shear force diagrams: Kocaeli earthquake Sakarya station flexural stiffness ratio: 0.90.....	46

Figure 5.24. Shear force diagrams: Kocaeli earthquake Yarimca station flexural stiffness ratio: 0.90.....	47
Figure 5.25. Moment diagrams: Kobe earthquake flexural stiffness ratio: 0.90	48
Figure 5.26. Moment diagrams: Kocaeli earthquake-Sakarya station flexural stiffness ratio: 0.90.....	48
Figure 5.27. Moment diagrams: Kocaeli earthquake-Yarimca station flexural stiffness ratio:0.90.....	49
Figure 6.1. C1, C2, C3 Coefficients according to angle of internal friction (API 2A,2000)	51
Figure 6.2. Modulus of subgrade reaction according to angle of internal friction (API 2A,2000).....	52
Figure 6.3. Typical multilinear plastic link property defined for p-y curve.....	53
Figure 6.4. Force-deformation definition and takeda hysteresis type (Sap2000 Link/Support Directional Properties)	54
Figure 6.5. Typical multilinear plastic link property defined for p-y curve, h=1 m.....	55
Figure 6.6. Typical multilinear plastic link property defined for p-y curve, h=2 m.....	55
Figure 6.7. Typical multilinear plastic link property defined for p-y curve, h=3 m.....	55
Figure 6.8. Typical multilinear plastic link property defined for p-y curve, h=4 m.....	56
Figure 6.9. Typical multilinear plastic link property defined for p-y curve, h=5 m.....	56
Figure 6.10. Typical multilinear plastic link property defined for p-y curve, h=6 m.....	56
Figure 6.11. Typical multilinear plastic link property defined for p-y curve, h=7 m.....	57

Figure 6.12. Typical multilinear plastic link property defined for p-y curve, h=8 m	57
Figure 6.13. Typical multilinear plastic link property defined for p-y curve, h=9 m	57
Figure 6.14. Typical multilinear plastic link property defined for p-y curve, h=10 m	58
Figure 6.15. Typical multilinear plastic link property defined for p-y curve, h=11 m	58
Figure 6.16. Typical multilinear plastic link property defined for p-y curve, h=12 m	58
Figure 6.17. Typical multilinear plastic link property defined for p-y curve, h=13 m	59
Figure 6.18. Typical multilinear plastic link property defined for p-y curve, h=14 m	59
Figure 6.19. Typical multilinear plastic link property defined for p-y curve, h=15 m	59
Figure 6.20. No soil embedment mode shape and period	60
Figure 6.21. 5m soil embedment mode shape and period	61
Figure 6.22. 10 m soil embedment mode shape and period	62
Figure 6.23. 15 m soil embedment mode shape and period	63
Figure 6.24. 5% damped elastic acceleration response spectra for Kobe earthquake vertical and horizontal component, the dots represent the spectral acceleration values for no soil embedment and 15 m soil embedment case modal periods	64
Figure 6.25. 5% damped elastic acceleration response spectra for Sakarya station Kocaeli earthquake vertical and horizontal component, the dots represent the spectral acceleration values for no soil embedment and 15 m soil embedment case modal periods	65
Figure 6.26. 5% damped elastic acceleration response spectra for Yarimca station Kocaeli earthquake vertical and horizontal component, the dots represent the spectral	

acceleration values for no soil embedment and 15 m soil embedment case modal periods	65
Figure 6.27. Shear force diagrams: Kobe earthquake	66
Figure 6.28. Shear force diagrams: Sakarya station- Kocaeli earthquake.....	66
Figure 6.29. Shear force diagrams: Yarimca station Kocaeli earthquake	67
Figure 6.30. Moment diagrams: Kobe earthquake.....	68
Figure 6.31. Moment diagrams: Sakarya Station Kocaeli earthquake.....	68
Figure 6.32. Moment diagrams: Yarimca station Kocaeli earthquake.....	69

LIST OF TABLES

Table 2.1. Pier geometric properties	5
Table 3.1. Characteristic of earthquake records.....	8
Table 4.1. Typical elastic modulus of soils based on soil type and density (“Engineering and Design Settlement Analysis” <i>U.S. Army Corps of Engineers</i> , Washington, DC 20314-1000, 1990)	19
Table 4.2. Vertical and horizontal stiffness values according to soil and foundation properties.	20
Table 4.3. Deck dimensions.....	21
Table 4.4. Total superstructure mass for k : 0.30 - 0.60 - 0.90.....	23
Table 5.1. Fundamental periods for flexural stiffness ratio 0.30 – 0.60 – 0.90.....	28
Table 5.2. Shear forces (kN) under horizontal and both vertical and horizontal ground motion for flexural stiffness ratios: 0.30 – 0.60 – 0.90.....	37
Table 5.3. Moment diagrams (kNm) under horizontal-only and both horizontal and vertical earthquake for flexural stiffness ratios 0.30 – 0.60 – 0.90	38

LIST OF SYMBOLS/ABBREVIATIONS

A	Foundation area
b	Foundation breadth
C_1, C_2, C_3	Coefficients according to angle of internal friction
d	Foundation width
D	Average column diameter in transverse direction
E	Young's modulus of elasticity
G	Shear Modulus of the Soil
H	Soil depth
H.M	Horizontal ground motion
k	Initial modulus of subgrade reaction
K_x	Horizontal stiffness of the soil
K_z	Vertical stiffness of the soil
l	Spacing of the vertical soil springs
L_{span}	Span Length
n	Mode number
p_u	Ultimate bearing capacity at depth
p_{us}	Ultimate resistance (shallow)
p_{ud}	Ultimate resistance (deep)
T_n	Natural period of vibration
V.M	Vertical ground motion
y	Lateral deflection
γ	Effective soil weight
φ'	Angle of internal friction of sand
ν	Poisons ratio of the soil

1. INTRODUCTION

1.1. Objectives of the Study

The vertical component of earthquake ground motion has generally been ignored in the earthquake resistant design of bridges. The structural design of a bridge is carried out under the effects of horizontal earthquake loads. Extensive damages of highway bridges in recent decades and during the recent earthquakes such as Loma Prieta (1989), North Rich (1994) and Kobe (1995) lead to new revisions in bridge design codes. These recent earthquakes and field evidences of their damaging effects show that the vertical component of the earthquake ground motion needs to be considered in the seismic design bridges.

In this thesis, two cases are studied. First is to investigate the effect of vertical component of earthquake records on a single column bridge pier. To study the effects of vertical ground motion on a bridge pier, a numerical model of 30 m-high single column pier with a 15 m-wide cap beam is subjected to horizontal and vertical ground motions recorded at Sakarya and Yarimca station of Kocaeli (1999) earthquake and Kobe (1995) earthquake. Three acceleration time histories for each principal direction are used to carry out dynamic analyses in Sap2000 to assume the response of the pier.

Second purpose of the study is to investigate the possible dynamic response of the bridge column exposed to the horizontal component of same ground motion records when the column is buried earth fill. Nowadays largescale construction projects in Turkey are required to be completed in the shortest possible time and with minimum cost. This leads the contractors to cut down each item. The distance of the dumping site during the construction phase may also be a waste of time for the contractor. Due to the lack of dumping area or being far from the construction site, excessive landfill material obtained from the excavations at large scale project is usually dumped under the bridge piers. This may cause the column to be buried under the soil up to half its height. This case which is not considered in the application project of the bridge may cause the column forces to increase. Seismic response of the column is examined in state of buried under 5 m, 10 m and 15 m of earth fill. The soil lateral force deformation relation is idealized by multi-linear plastic p-y curves.

1.2. Literature Review

Many works have been published concerning structural design that takes into account horizontal earthquake loads but the vertical component of earthquake ground motion has commonly been underestimated in earthquake design of structures.

Papazoglou and Elnashai (1996) defined some site data and results from dynamic analysis on possible structural effects of strong vertical ground motion. They attempted to collate site data and evaluate dynamic analysis results on possible structural effects of strong vertical ground motion. Papazoglou and Elnashai (1996) documented observational data from three different earthquakes and checked them regarding to failure modes of buildings and bridges owing to high vertical earthquake forces. They validated structural failure can be caused by direct tension or compression, together with the effect of vertical motion on shear and flexural response. Papazoglou and Elnashai (1996) indicated that high vertical to horizontal ground motion ratios caused severe failures at the 1986 Greek earthquake and the Northridge earthquake.

In 2003, Rahai introduced an existing four-span RC bridge model that was analyzed using the finite element method. Different accelerograms under horizontal ground or simultaneously to the horizontal and vertical ground motion were used in the dynamic linear and nonlinear time history analysis of the model. Axial and shear forces, longitudinal displacements in the piers are studied. Results showed significant variability in axial force values that could result in circumferential cracks in the RC piers (Rahai, 2004).

Kunnath (2008) researched the effects of vertical accelerations near the faults on the overall response of highway bridges. Nonlinear analysis models of an existing bridge in California were considered in the analytical study. For the analytical study, an extensive set of ground motions with horizontal PGA exceeding 0.5 g were chosen. At first stage, the analysis models were subjected to the horizontal components of the motion, in the second stage the bridge models were subjected to both horizontal and vertical components simultaneously. The analysis results revealed that vertical ground motions can have a significant effect on (i) moment demands at the middle of the span; (ii) demands at the face of the bent cap, and (iii) axial force demand in columns.

Kim, S. J., Holub, C. J. & Elnashai, A. S., (2008) investigated the effect of vertical ground motion on bridge piers for different ratios of peak vertical to horizontal (V / H) ground accelerations and the lapse of time between horizontal and vertical peak accelerations. The most important result is when the V / H ratio increases the axial force increases significantly within the piers. The studies additionally indicates that despite the fact pier moment differs due to vertical ground motion, high axial force variation causes the increase of the moment demand in some records even if the lateral displacement is decreased. It was also noted that the significant increase in variation of axial force corresponding to vertical ground motions led to an observed reduction of shear capacity up to 30%. The arrival time of the ground acceleration has a slightest effect in terms of shear demand and axial force variation for the considered structures and motion set. In perspective of the observations mentioned above, RC bridge piers that are exposed to the combined horizontal and vertical components of earthquakes may be more susceptible than those exposed to horizontal component of ground motion.

Kim, S. J., Holub, C. J. & Elnashai, A. S., (2011) presented a paper on the analysis of the effect of vertical earthquake ground motion on reinforced concrete bridge piers. Two separate bridges designed according to Federal Highway concept and damaged during the Northridge earthquake were examined. The effects of a series of ground motion records on the two bridges with different vertical to horizontal peak acceleration ratios were compared with the horizontal component of the earthquake. Additionally, the consequences of the arrival-time interval between horizontal and vertical peaks of acceleration were stated and compared with the case of coincident motion. The inclusion of the vertical component of ground motion was noted to have a significant effect on the response at all levels and on all components. Therefore, it was found that vertical motion should be involved in the evaluation and design analysis, particularly when there are no specific difficulties that interfere its inclusion.

2. DESCRIPTION OF SINGLE COLUMN BRIDGE PIER

In this study a reinforced concrete, single pier with a height of 30 m is examined under the horizontal and vertical components of the earthquake ground motions. The cap beam is inclined in transverse direction as it belongs to a conventional bridge at curb. The conventional bridges are constructed with movement joints and connections in contrast to integral bridges that has continuous decks. The column is oval shape filled section with a dimension of 2.5 m x 4.0 m and it is monolithically connected with the cap beam on top of it. The length of the column is 4 m in transverse direction. The cap beam has a height of 1.5 m with a reverse T section and a total width of 15 m. The cantilever part of the cap beam is 5.35 m from the outer surface of the column.

The pier bent supports a superstructure consisting of 11 prestressed reinforced concrete I shape beams on the left and right spans and a 25 cm thick reinforced concrete deck. There are 1.5 m width pavements on the left and right side of the deck. Under each prestressed beam, there are elastomeric bearings and are seated on the cap beam by means of these bearings. The horizontal displacement of the elastomeric bearings in the transverse direction are fixed by the shear keys on the cap beam, thereby prestressed beams are prevented from falling when exposed to horizontal ground motion at transverse direction. The single pier bent supports a 35 m of span length. I shape prestressed beams with a height of 150 cm are sufficient to carry the superstructure loads of 35 m-span. Properties of the single pier can be seen in Table 2.1. Section drawings and 3D view of the pier and the cap beam can be seen in Figure 2.1-Figure 2.3.

Table 2.1. Pier geometric properties.

Column Height	30 m
Cap Beam Width	15 m
Column Section	2.5 m x 4.0 m gross section
Cap Beam Section	Reverse T Section
Cap Beam Height	1.50 m
Number of Prestressed I girders over cap beam	11
Deck Thickness	25 cm
Left Pavement Width	1.50 m
Right Pavement Width	1.50 m
Span Length	35 m

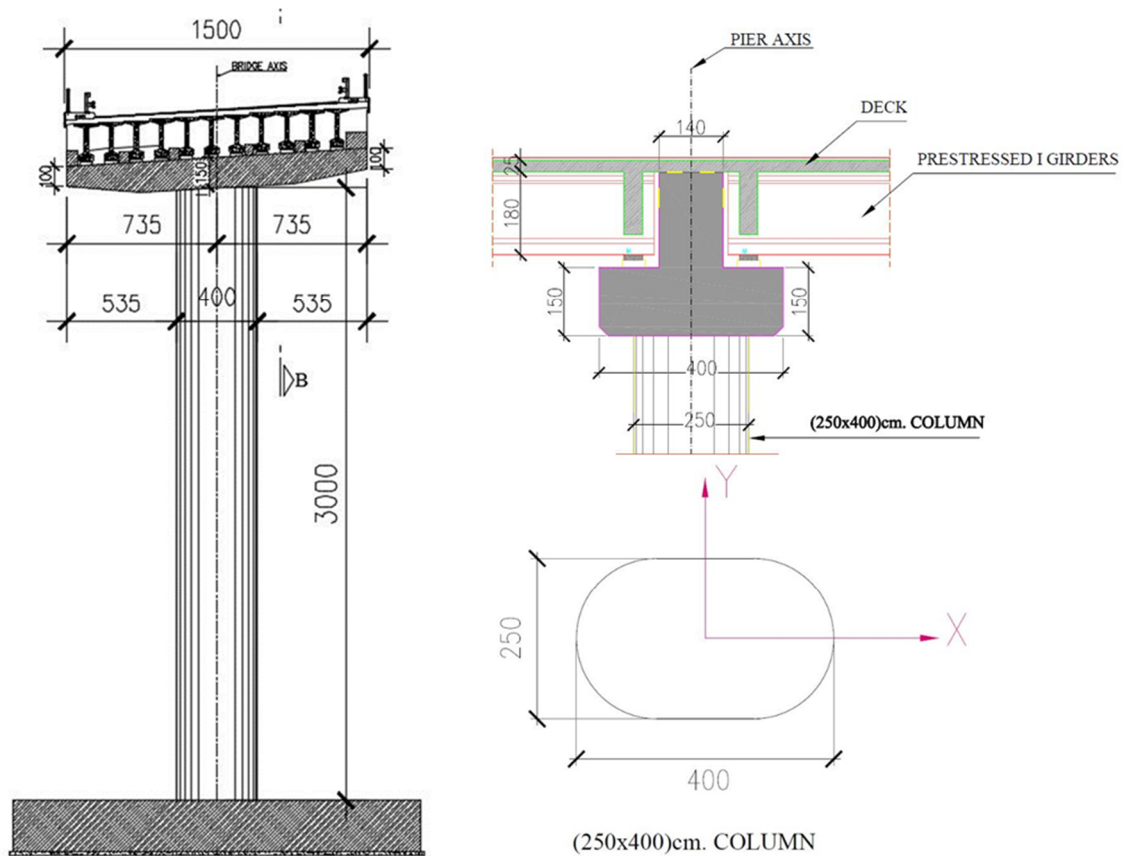


Figure 2.1. Section drawings of the pier and the cap beam.

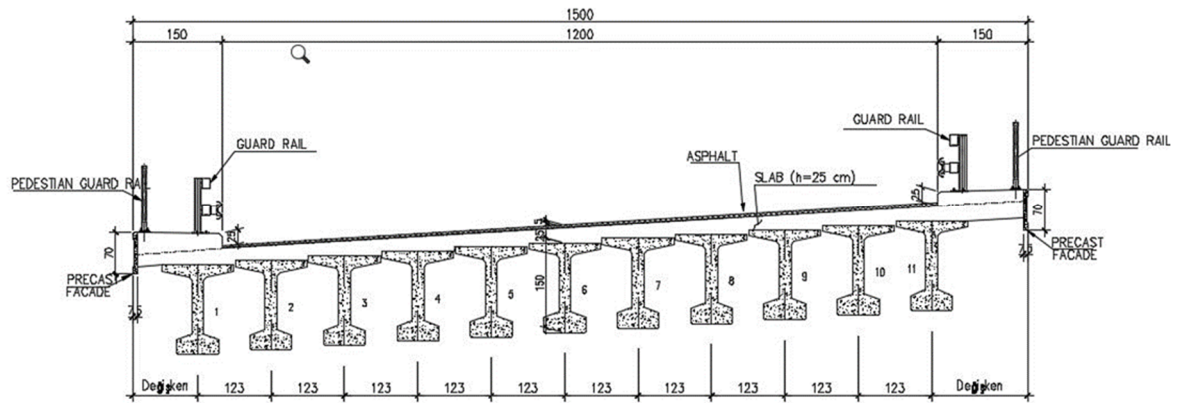


Figure 2.2. Superstructure cross section.

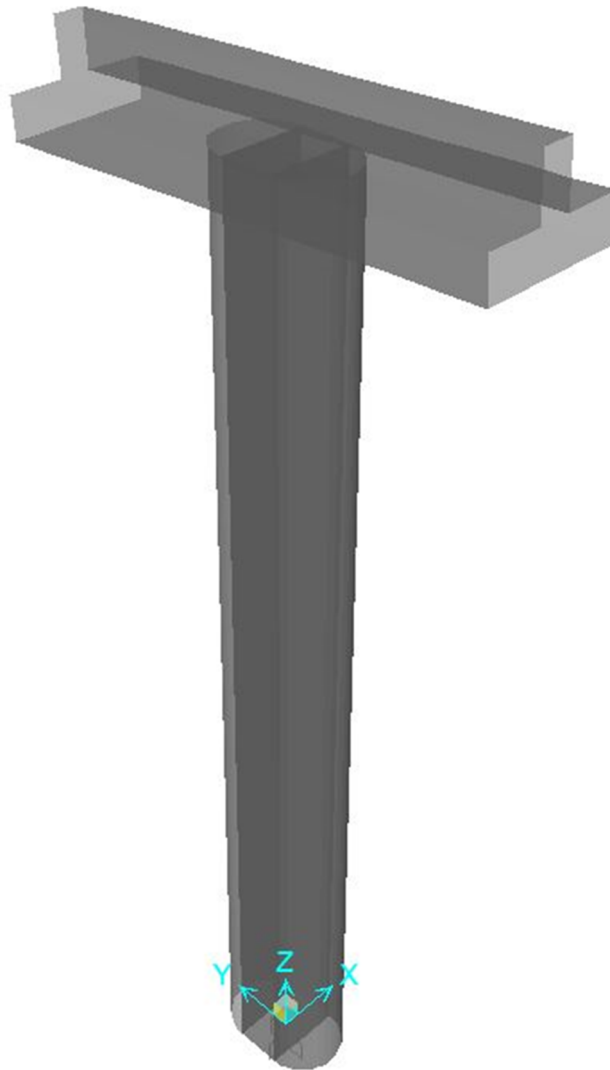


Figure 2.3. 3D view of the pier and the cap beam.

2.1. Material Properties of Pier

2.1.1. Concrete Properties

Column and cap beam characteristic strength of the concrete is taken as 30 MPa; whereas as can be seen in the Figure 2.4. For both members, the tensile strength of concrete is taken as zero. Modulus of elasticity of concrete is calculated by the formula given in AASHTO LRFD 5.4.2.4

Compression Strength,	$f'_c = 30 \text{ MPa}$
Elastic Modulus,	$E_c = 4730 (f'_c)^{0.5}$

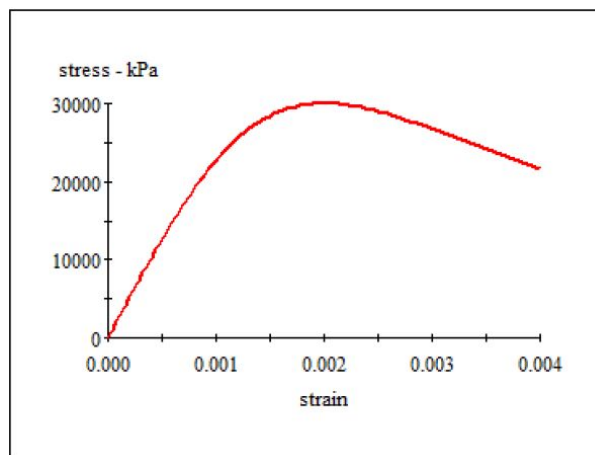


Figure 2.4. Stress-strain diagram for concrete.

3. SELECTION OF GROUND MOTION RECORDS

In this study, real earthquake records are used. The time history analysis is done using Kobe (1995-station: Port Island), Kocaeli (1999-station:Yarimca), Kocaeli (1999-station:Sakarya) records under two loading condition: first with horizontal motion, and second under horizontal and vertical motions. The records characteristics are shown in table 3.1.

Table 3.1. Characteristic of earthquake records.

Records	Date	M_s	Peak of H.M	Peak of V.M
Kobe-Port Island	1995	7.2	0.29g	0.57g
Kocaeli-Yarimca	1999	7.51	0.32g	0.24g
Kocaeli-Sakarya	1999	7.4	0.39g	0.23g

Elastic acceleration response spectra of horizontal ground motions and vertical ground motions are obtained using Seismosignal software.

3.1. Horizontal Acceleration, Velocity, Displacement Time Histories Kobe (Jan. 16, 1995)- Station: Port Island

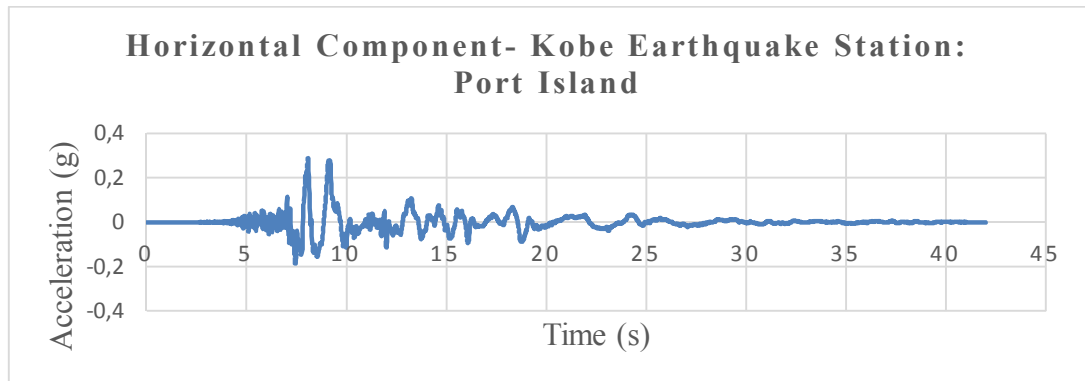


Figure 3.1. Horizontal acceleration time history Kobe (Jan. 16, 1995) - station: Port Island.

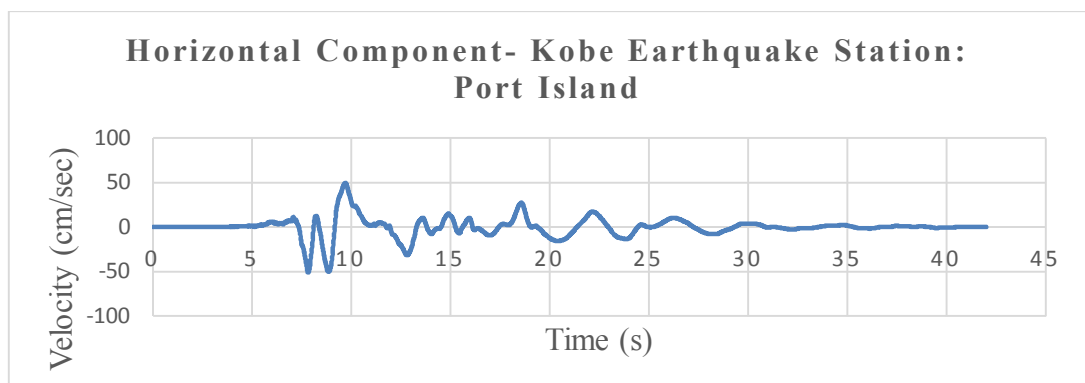


Figure 3.2. Horizontal velocity time history Kobe (Jan. 16, 1995) - station: Port Island.

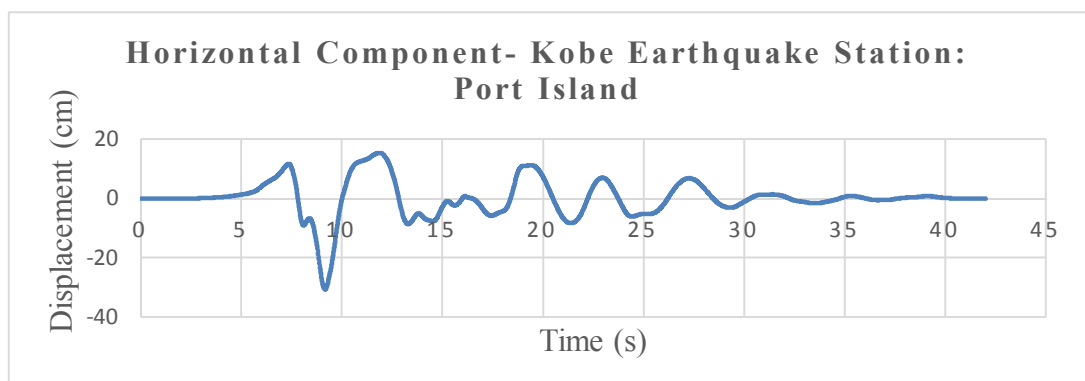


Figure 3.3. Horizontal displacement time history Kobe (Jan. 16, 1995) - station: Port Island.

3.2. Vertical Acceleration, Velocity, Displacement Time Histories Kobe (Jan. 16, 1995)- Station: Port Island

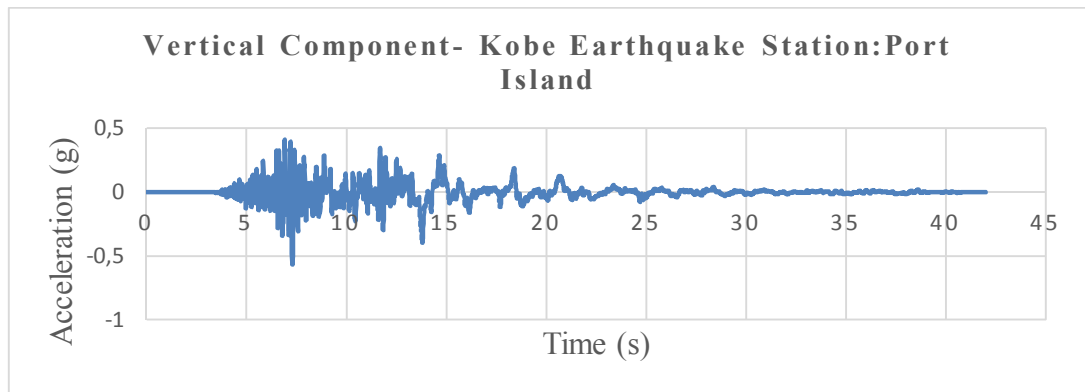


Figure 3.4. Vertical acceleration time history Kobe (Jan. 16, 1995) - station: Port Island.

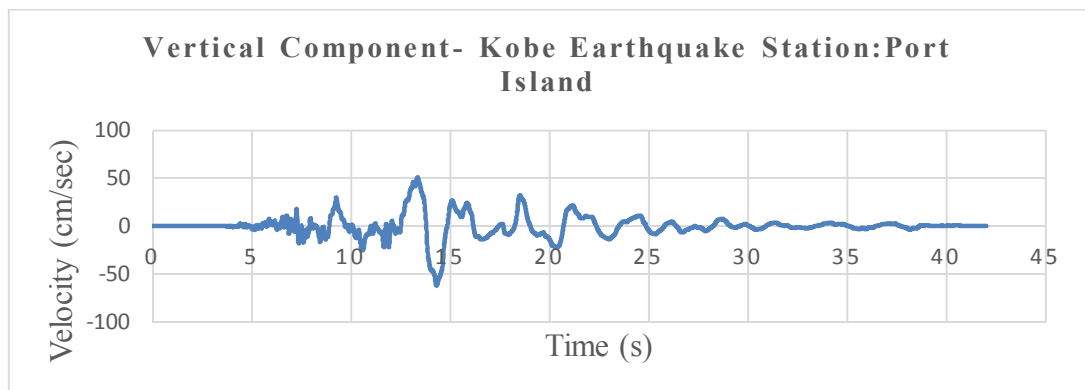


Figure 3.5. Vertical velocity time history Kobe (Jan. 16, 1995) - station: Port Island.

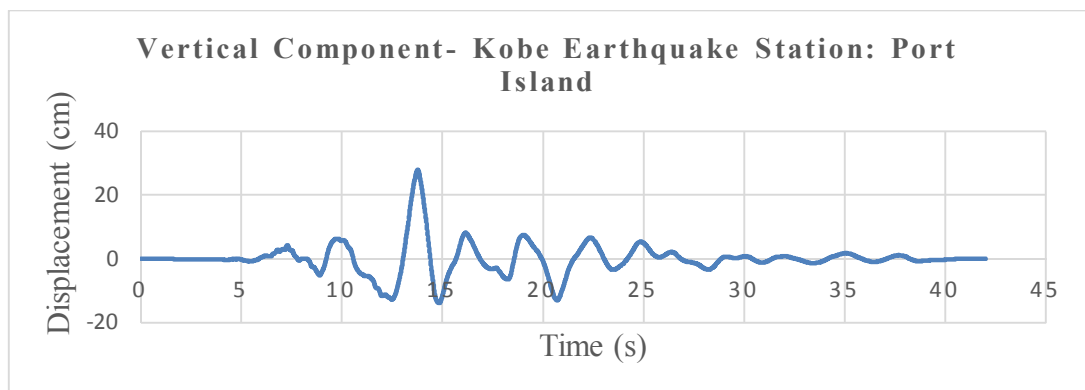


Figure 3.6. Vertical displacement time history Kobe (Jan. 16, 1995) - station: Port Island.

3.3. Horizontal Acceleration, Velocity, Displacement Time Histories Kocaeli (Aug. 17, 1999) - Station: Sakarya

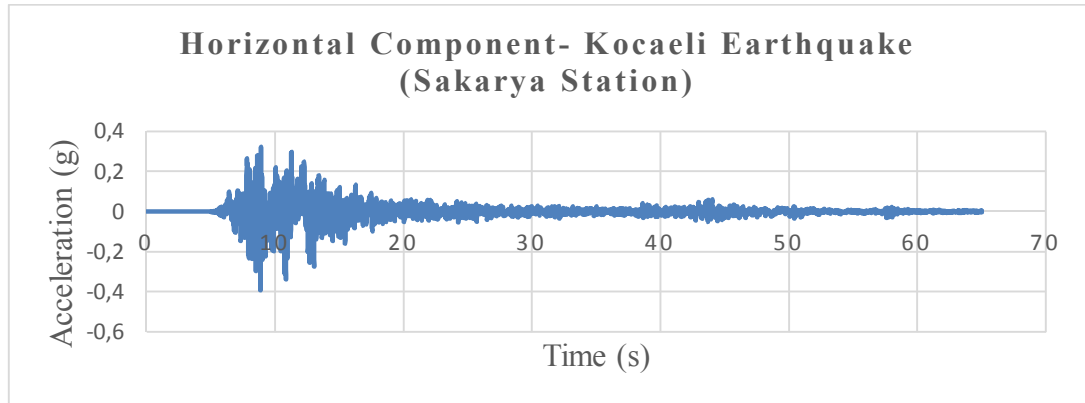


Figure 3.7. Horizontal acceleration time history Kocaeli (Aug. 17, 1999) - station: Sakarya.

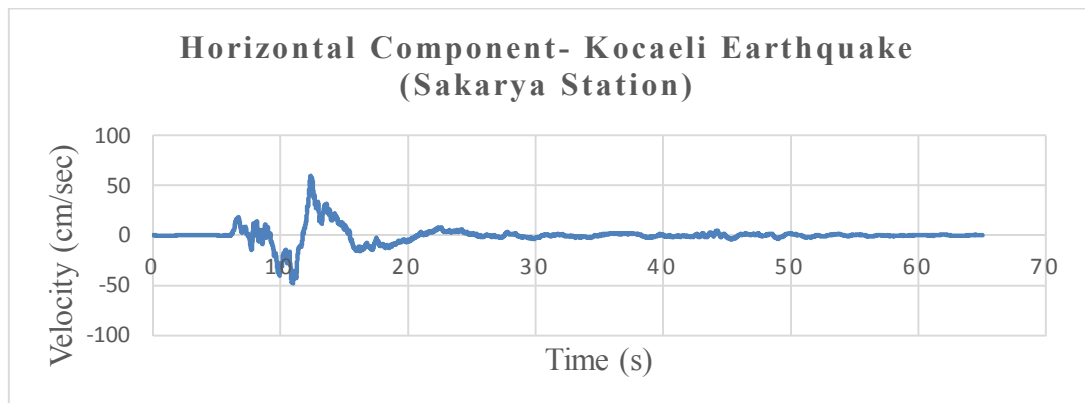


Figure 3.8. Horizontal velocity time history Kocaeli (Aug. 17, 1999) - station: Sakarya.

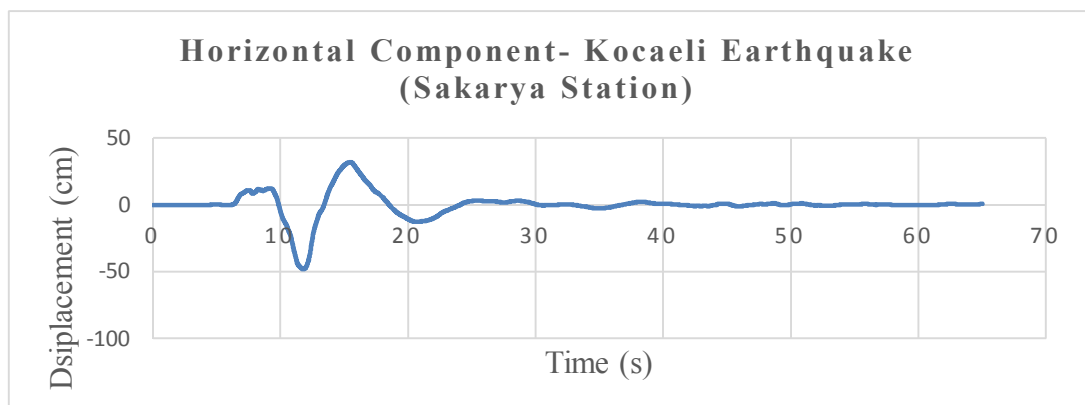


Figure 3.9. Horizontal displacement time history Kocaeli (Aug. 17, 1999)-station: Sakarya.

3.4. Vertical Acceleration, Velocity, Displacement Time Histories Kocaeli (Aug. 17, 1999) - Station: Sakarya

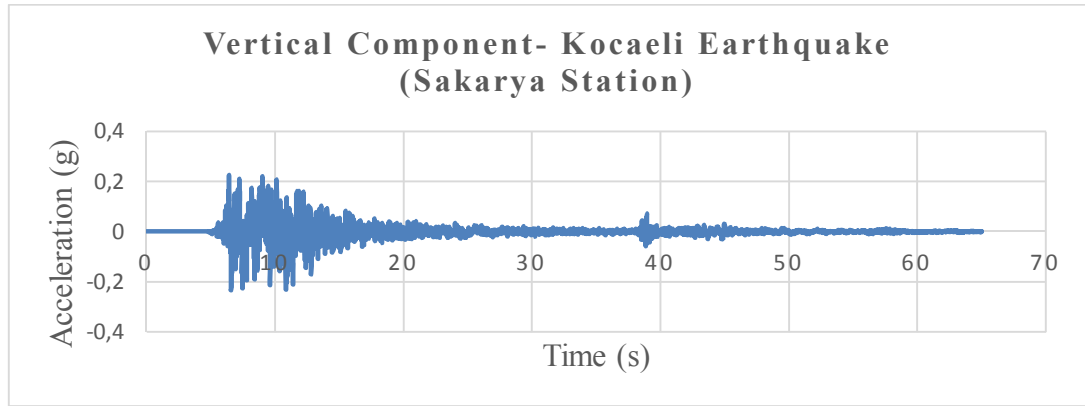


Figure 3.10. Vertical acceleration time history Kocaeli (Aug. 17, 1999) - station: Sakarya.

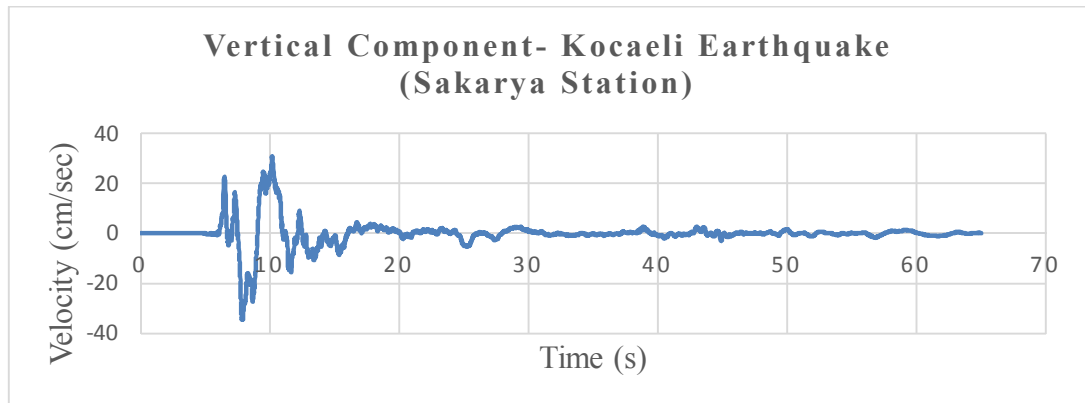


Figure 3.11. Vertical velocity time history Kocaeli (Aug. 17, 1999) - station: Sakarya.

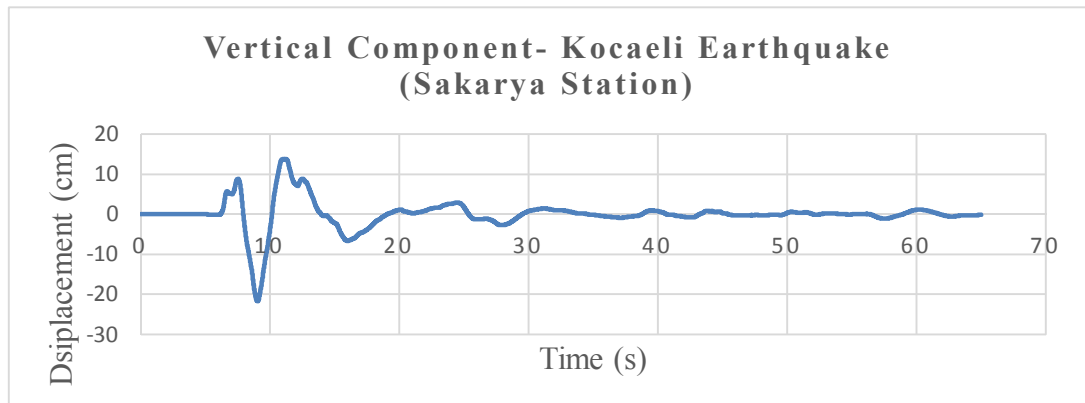


Figure 3.12. Vertical displacement time history Kocaeli (Aug. 17, 1999) - station: Sakarya.

3.5. Horizontal Acceleration, Velocity, Displacement Time Histories Kocaeli (Aug. 17, 1999) - Station: Yarimca150

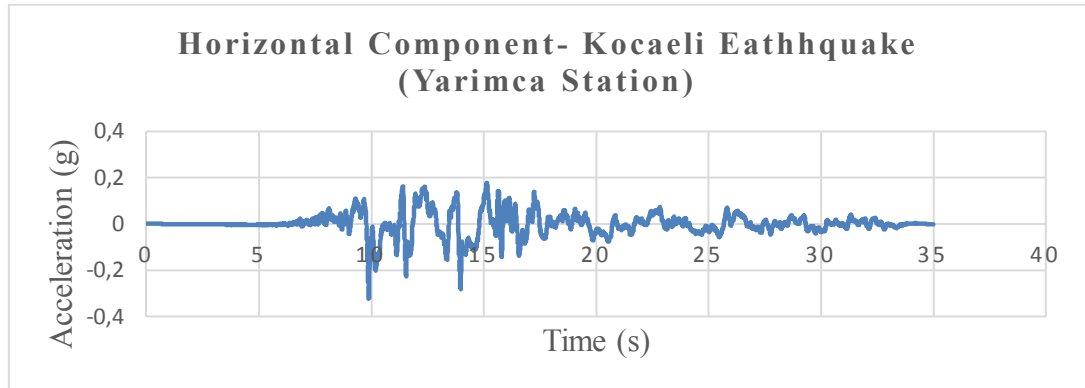


Figure 3.13. Horizontal acceleration time history Kocaeli (Aug.17, 1999) - station: Yarimca150.

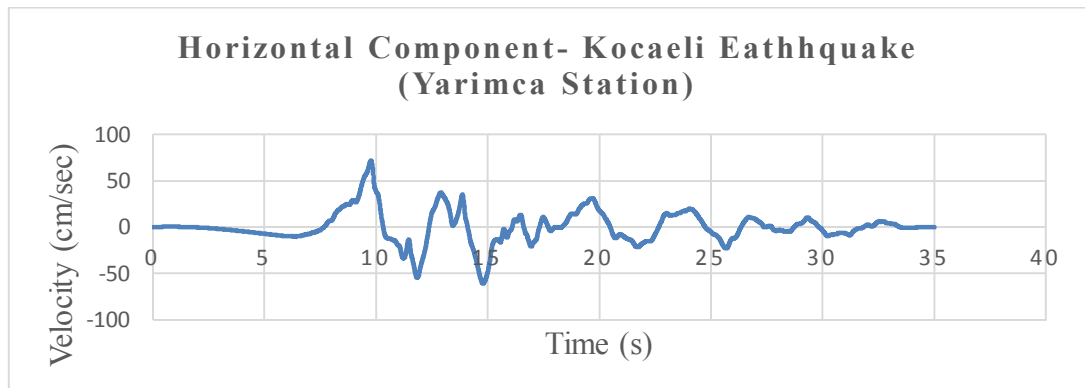


Figure 3.14. Horizontal velocity time history Kocaeli (Aug.17, 1999)-station: Yarimca150.

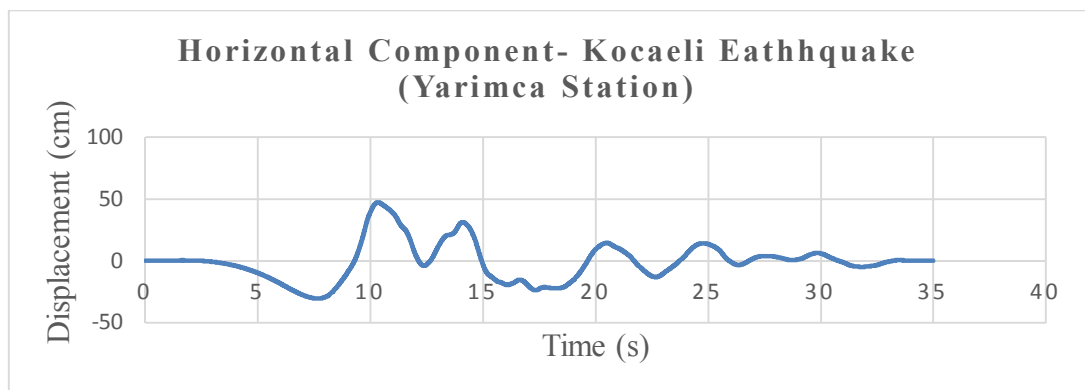


Figure 3.15. Horizontal time history Kocaeli (Aug.17, 1999)-station Yarimca150.

3.6. Vertical Acceleration, Velocity, Displacement Time Histories Kocaeli (Aug. 17, 1999)- Station: Yarimca150

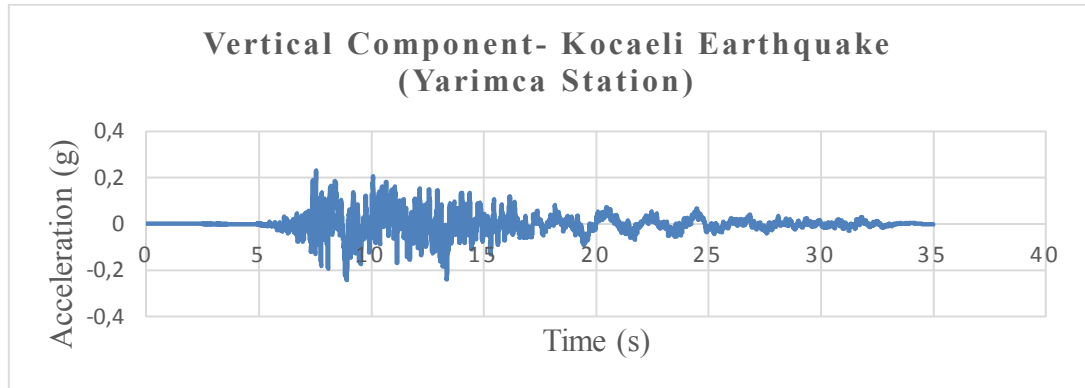


Figure 3.16. Vertical acceleration time history Kocaeli (Aug.17,1999)-station: Yarimca150.

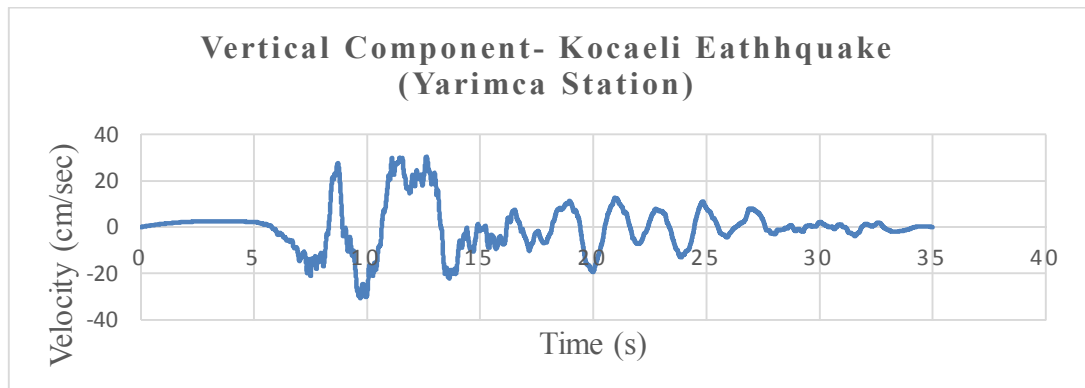


Figure 3.17. Vertical velocity time history Kocaeli (Aug.17, 1999)-station: Yarimca150.

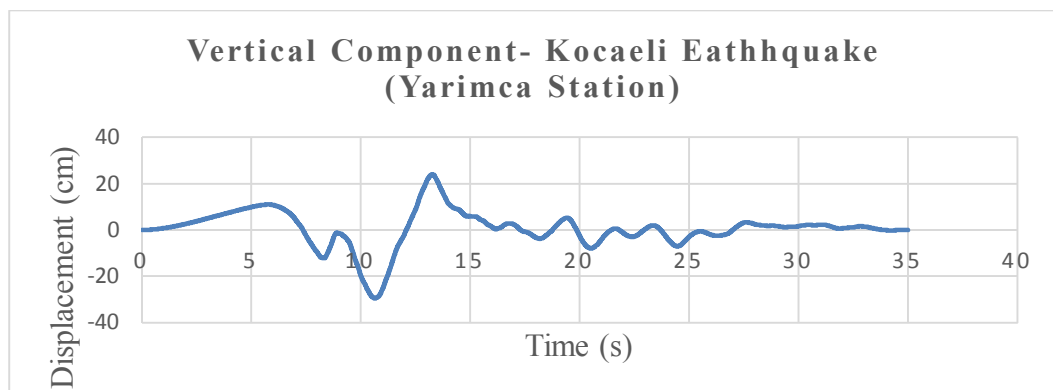


Figure 3.18. Vertical time history Kocaeli (Aug.17, 1999)-station Yarimca150.

4. STRUCTURAL MODELLING

The structure is transformed into a mathematical model by finite element method. The finite element method is used for this study by means of the Sap2000 Structural Analysis Software. To simplify the single column bent, the column and the cap beam are modeled as frame elements that acquire the properties of the sections. Cap beam and column are monolithically connected to each other. Cross-sections of the column and the cap beam are generated in SAP2000 section designer. Analysis have been done in the transverse direction of the pier.

4.1. Structural Model for Investigating the Effects of Vertical Component of Earthquake

Two different mathematical models were created which the column is subjected to the horizontal-only component of the earthquake and both the horizontal and vertical component of the earthquake.

Mathematical Model used in case of horizontal earthquake effect, cap beam was added to the model as a lumped mass over the top of the column, whereas, mathematical model used in case of both horizontal and vertical earthquake effected, cap beam was added to the model as frame element with its own cross section properties. Typical mathematical model used in case of horizontal earthquake effect is seen in Figure 4.4. and typical mathematical model use in case of both horizontal and vertical earthquake effect is seen in Figure 4.5. Structural models are constructed for three different flexural stiffness ratios of cap beam and column: 0.30-0.60-0.90. The depth of the cap beam is modified to provide this flexural stiffness ratio. Pier cap beam flexural stiffness ratios 0.30-0.60-0.90 are shown in Figure 4.1., Figure 4.2, Figure 4.3. respectively.

Linear elastic link elements are used instead of fixed supports to idealize the soil stiffness for mathematical models. Thus, the shallow foundation at the base of the column is considered to make rotation and displacement in the earthquake direction. The equations

used to calculate the spring stiffness values applied at the supports of the pier are given in chapter 4.1.1.

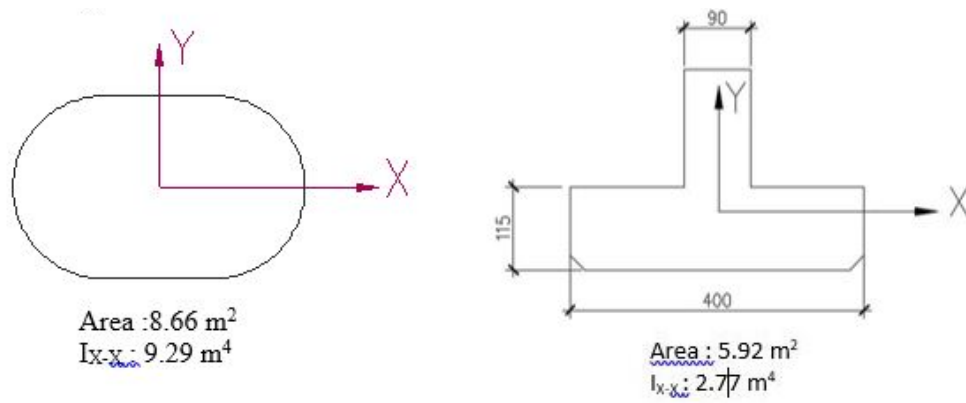


Figure 4.1. Pier-cap beam flexural stiffness ratio: 0.30.

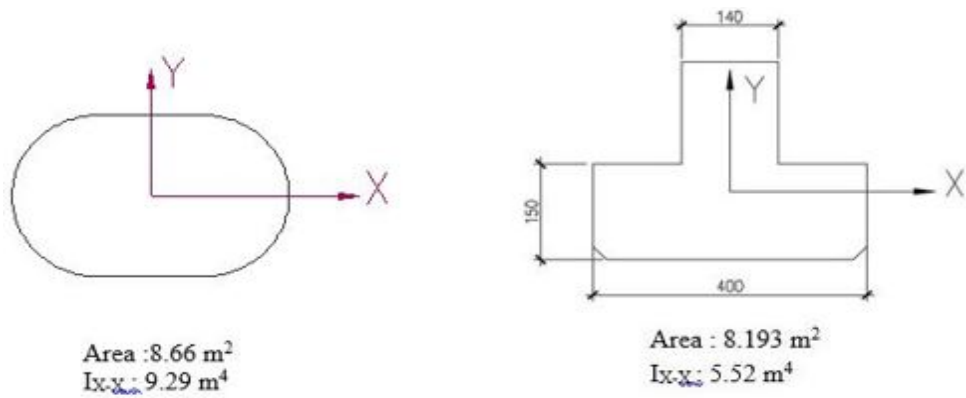


Figure 4.2. Pier-cap beam flexural stiffness ratio: 0.60.

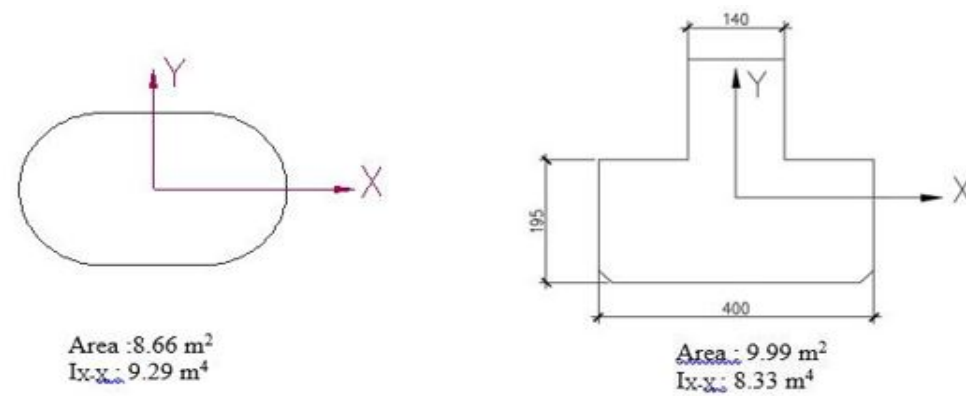


Figure 4.3. Pier-cap beam flexural stiffness ratio: 0.90.

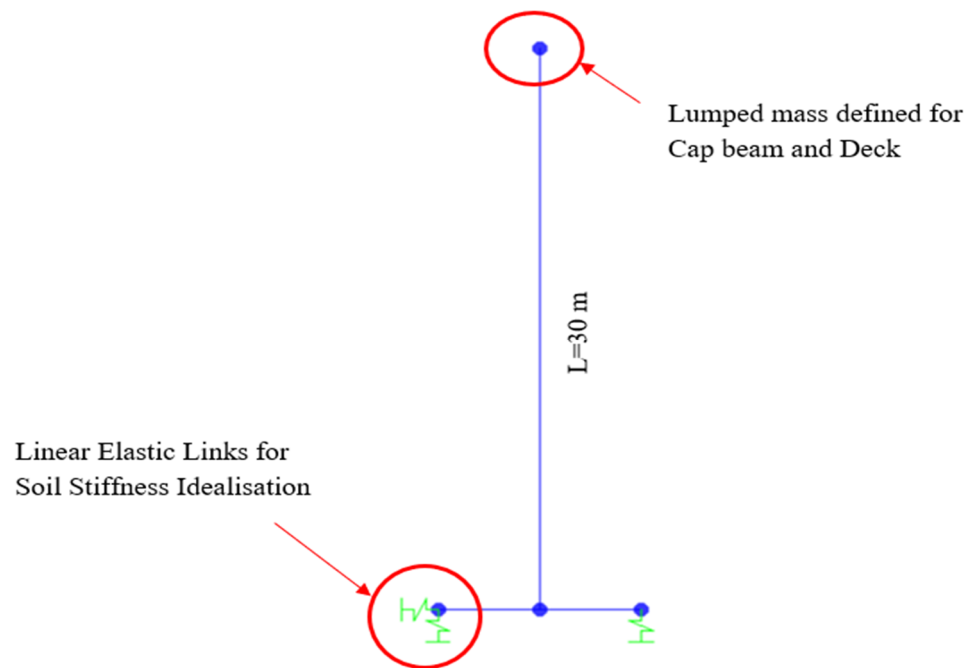


Figure 4.4. Lumped mass analysis model.

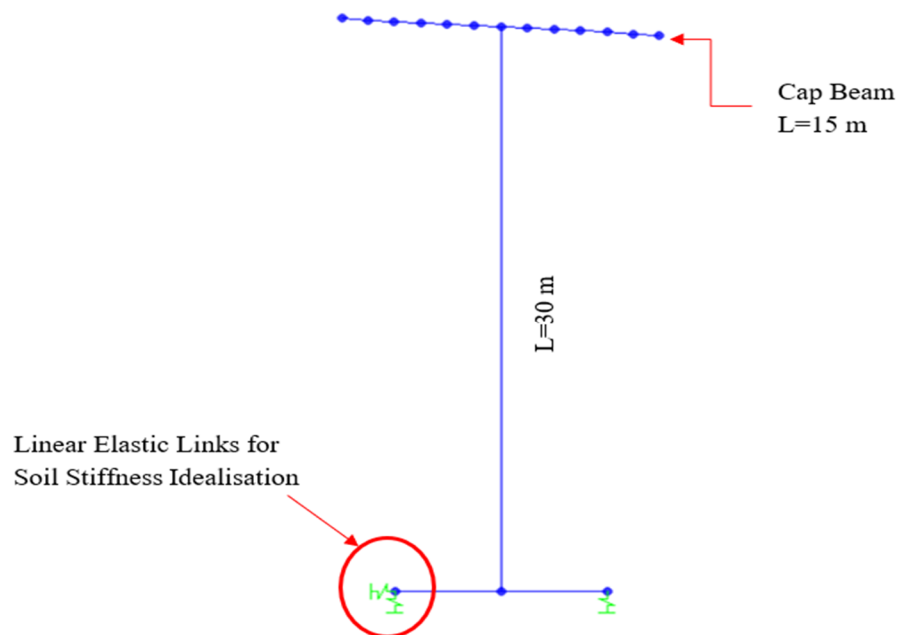


Figure 4.5. Distributed mass analysis model.

4.1.1. Soil Stiffness Idealization

The spring stiffnesses are applied on the supports of the pier instead of fixed supports. Springs are introduced at the base of the column to incorporate soil-foundation flexibility. Displacement and rotation of supports under horizontal and vertical loads can create additional P- Δ effect due to cantilever part of the cap beam. Therefore, it is important to give great importance to the assessment of the stiffnesses of the supports and foundations as to the stiffnesses of the deck structure.

Soil stiffness is modelled using simple linear springs. Estimation for the stiffnesses of the shallow footing foundations shown in Figure 10.1. obtained with the equations based on the theory for an elastic half space. The equations given by Lambe, T.W. (1969) and Richard, F.E. (1970) are simplified by Hambly (1994) The spring stiffnesses suggested by Hambly (1994) applied to the structure are given below.

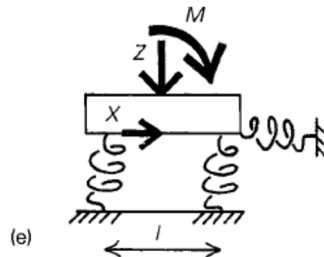


Figure 4.6. Vertical and horizontal stiffness of the shallow foundation.

14mx14m shallow rectangular foundation is assumed under single column. Hambly (1994) suggests to present the vertical and rotational stiffness by two parallel springs spaced l apart as shown in Figure 4.6.

The spacing of the vertical springs are given by the equation below:

$$l = 0.82b^{0.25}d^{0.75} \quad (4.1)$$

$$l = 0.82 \times 12^{0.25} \times 12^{0.75} = 9.84 \text{ m}$$

Shear Modulus of the soil G , vertical stiffness of the soil K_z , and the horizontal stiffness of the soil K_x , is given by the following formulas:

$$G = \frac{E}{[2(1 + \nu)]} \quad (4.2)$$

$$K_z = \frac{2.5GA^{0.5}}{(1 - \nu)} \quad (4.3)$$

$$K_x = 2G(1 + \nu)A^{0.5} \quad (4.4)$$

where;

ν = Poissons ratio of the soil, generally lie in range 0.3-0.5

A =foundation area = bd

E =Young modulus of the soil

The value of young modulus of the soil is taken as 15000 kN/m² (160 tsf) for loose sand which is based on USACE (U.S Army Corps of Engineers), Settlement Analysis.

Table 4.1. Typical elastic modulus of soils based on soil type and density (“Engineering and Design Settlement Analysis” *U.S. Army Corps of Engineers*, Washington, DC 20314-1000, 1990).

Soil	E_s , tsf
Clay	
Very soft clay	5 - 50
Soft clay	50 - 200
Medium clay	200 - 500
Stiff clay, silty clay	500 - 1000
Sandy clay	250 - 2000
Clay shale	1000 - 2000
Sand	
Loose sand	100 - 250
Dense sand	250 - 1000
Dense sand and gravel	1000 - 2000
Silty sand	250 - 2000

Table 4.2. Vertical and horizontal stiffness values according to soil and foundation properties.

Foundation breadth, b	12 m
Foundation width, d	12 m
Young's modulus of soil, E	15000 kN/m ²
Shear Modulus of Soil, G	5357 kN/m ²
Vertical Stiffness, K_z	267857 kN/m
Horizontal Stiffness, K_x	180000 kN/m

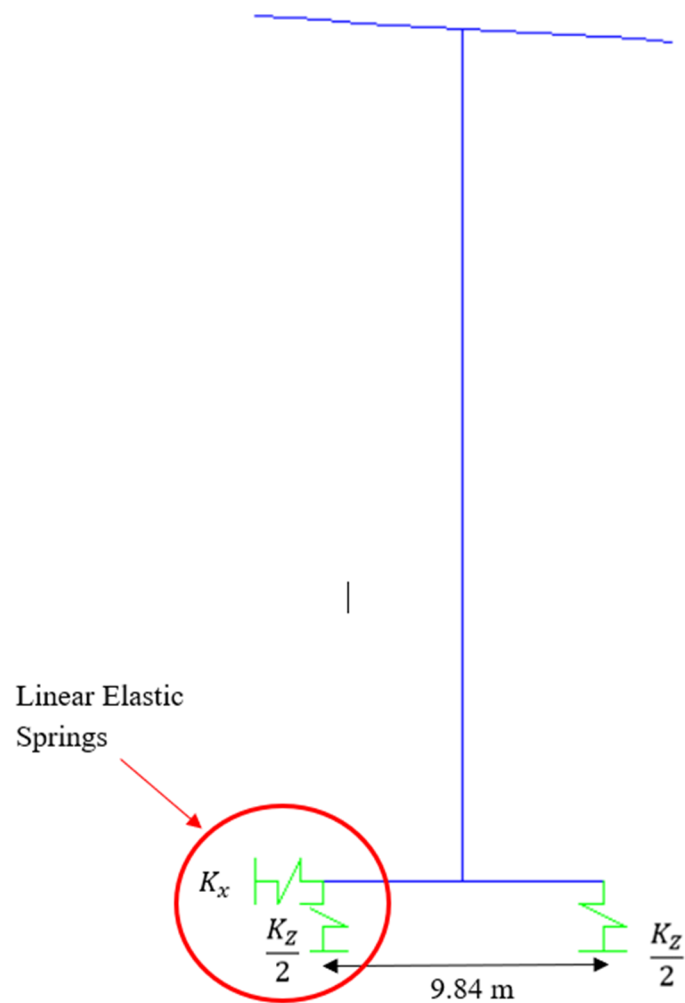


Figure 4.7. Analysis model with linear elastic springs represents the vertical and horizontal stiffness of shallow footing foundation.

4.1.2. Loads Considered for Pier

The superstructure consists of 11 contiguous prestressed I shape beams with a height of 180 cm. The area of each prestressed beam is 0.607 m^2 . The total width of the slab with 25 cm thickness over the girders is 15 m. The deck consists of 12 m wide traffic way and 1.5 m left and right pavements. Superstructure dimensions is given in Table 4.3. Superstructure cross section is given in Figure 4.8. Total superstructure mass for each cap beam / column flexural stiffness ratios k : 0.30-06.0-0.90 are given in Table 4.4.

Table 4.3. Deck dimensions.

Area of Prestressed I shape Girder	0.607 m^2
Number of prestressed Girders	11
Slab Thickness	0.25 m
Slab Width	15 m
Pavement Thickness	0.25 m
Pavement Width	3 m
Asphalt Thickness	0.08 m
Asphalt Width	12 m

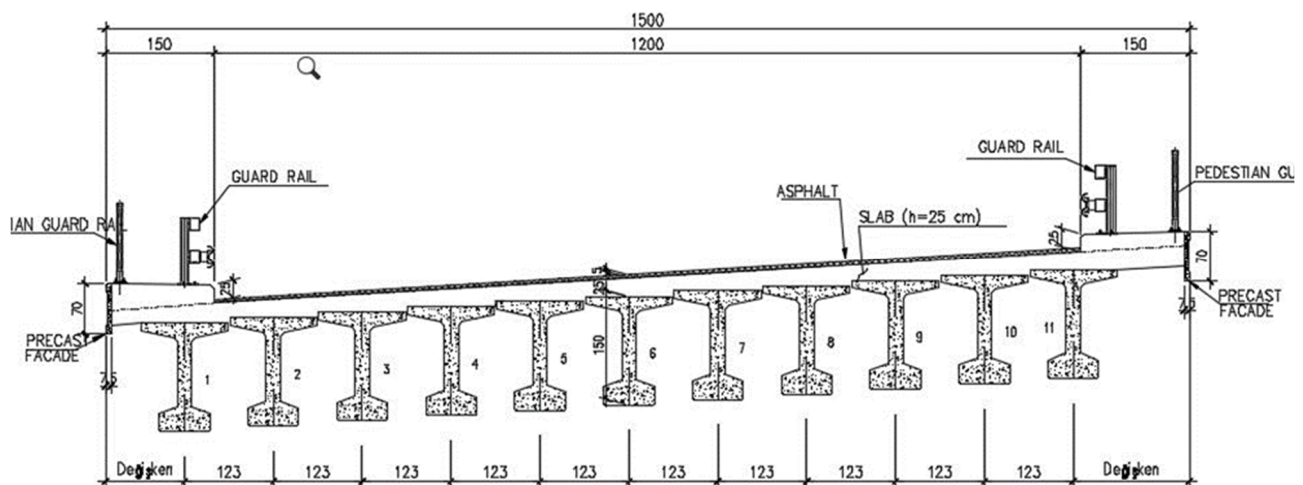


Figure 4.8. Superstructure cross section.

Total deck load and mass for 35 m span length are calculated below.

$$\text{Asphalt load} = 0.08 \text{ m} \times 12 \text{ m} \times 25 \text{ kN/m}^3 = 24 \text{ kN/m}$$

$$\text{Sidewalk load} = 0.25 \text{ m} \times (1.5 + 1.5) \text{ m} \times 25 \text{ kN/m}^3 = 18.75 \text{ kN/m}$$

$$\text{Slab load} = 0.25 \text{ m} \times 15 \text{ m} \times 25 \text{ kN/m}^3 = 93.75 \text{ kN/m}$$

$$\text{Girder load} = 0.6071 \text{ m}^2 \times 11 \times 25 \text{ kN/m}^3 = 166.95 \text{ kN/m}$$

$$\text{Total deck load} = 303.45 \text{ kN/m} \times 35 \text{ m} = 10620.8 \text{ kN}$$

$$\text{Total deck mass} = 10620.8 \text{ kN} / 9.81 = 1082.65 \text{ ton}$$

Cap beam is assigned as lumped mass in analysis model with horizontal component of earthquake. Mass values are as follows for $k=0.30$, $k=0.60$, $k=0.90$ flexural stiffness ratios respectively.

Cap beam mass for $k=0.30$ flexural stiffness ratio (column/cap beam):

$$\text{Cap beam area} = 1.14 \text{ m} \times 4 \text{ m} + 2 \text{ m} \times 1.40 \text{ m} = 6.4 \text{ m}^2$$

$$\text{Cap beam width} = 14.7 \text{ m}$$

$$\text{Cap beam weight} = 14.7 \text{ m} \times 6.4 \text{ m}^2 \times 25 \text{ kN/m}^3 = 2352 \text{ kN}$$

$$\text{Cap beam mass} = 2352 \text{ kN} / 9.81 = 239.76 \text{ ton}$$

Cap beam mass for k=0.60 flexural stiffness ratio (column/cap beam):

$$\text{Cap beam area} = 1.5 \text{ m} \times 4 \text{ m} + 2 \text{ m} \times 1.40 \text{ m} = 8.8 \text{ m}^2$$

$$\text{Cap beam width} = 14.7 \text{ m}$$

$$\text{Cap beam weight} = 14.7 \text{ m} \times 8.8 \text{ m}^2 \times 25 \text{ kN/m}^3 = 3234 \text{ kN}$$

$$\text{Cap beam mass} = 3234 \text{ kN} / 9.81 = 329.66 \text{ ton}$$

Cap beam mass for k=0.90 flexural stiffness ratio (column/cap beam):

$$\text{Cap beam area} = 1.95 \text{ m} \times 4 \text{ m} + 2 \text{ m} \times 1.40 \text{ m} = 10.6 \text{ m}^2$$

$$\text{Cap beam width} = 14.7 \text{ m}$$

$$\text{Cap beam weight} = 14.7 \text{ m} \times 10.6 \text{ m}^2 \times 25 \text{ kN/m}^3 = 3895.5 \text{ kN}$$

$$\text{Cap beam mass} = 3895.5 \text{ kN} / 9.81 = 397.09 \text{ ton}$$

Table 4.4. Total superstructure mass for k: 0.30 - 0.60 - 0.90.

Flexural stiffness ratio k:0.30	1082.65 ton + 239.76 ton	1322.41 ton
Flexural stiffness ratio k:0.60	1082.65 ton + 329.66 ton	1412.32 ton
Flexural stiffness ratio k:0.90	1082.65 ton + 397.76 ton	1479.75 ton

4.1.3. Lumped Mass Analysis Model



Figure 4.9. Analysis model flexural stiffness ratio (cap beam / column) =0.30.

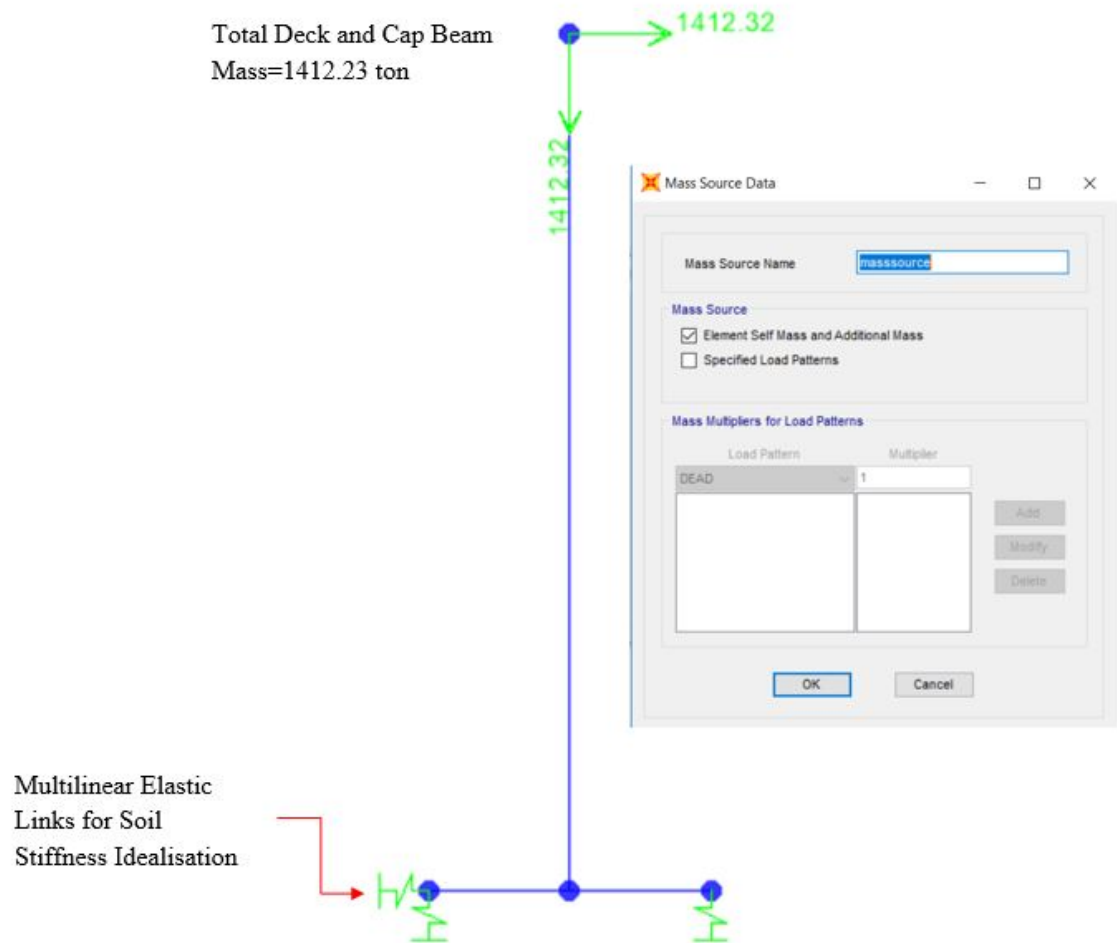


Figure 4.10. Analysis model flexural stiffness ratio (cap beam / column) = 0.60.

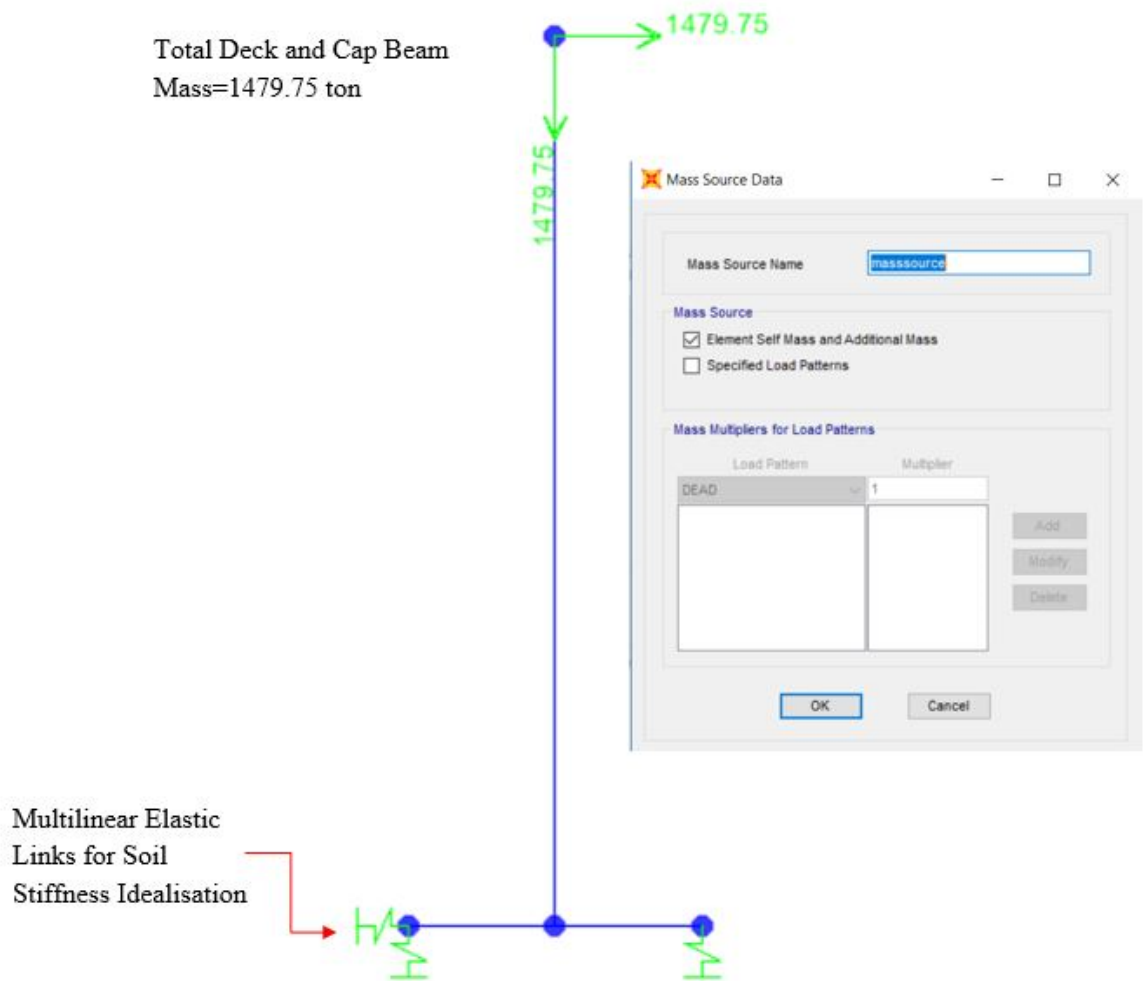


Figure 4.11. Analysis model flexural stiffness ratio (cap beam / column) = 0.90.

4.1.4. Distributed Mass Analysis Model

The total superstructure load will be transferred to the single column Pier through the elastomeric bearings. There are 11 elastomeric bearings under each prestressed beam. For this reason, the total superstructure load is divided into 11 and effected to the cap beam from these points. Superstructure load on the Pier bent can be seen in Figure 4.12.

$$\text{Total deck Load} = (303.45 \text{ kN/m}) * L_{\text{span}} = (303.45 \text{ kN/m}) * 35 \text{ m}$$

$$\text{Total deck Load} = 10620.84 \text{ kN}$$

$$\text{Total deck Load/Bearing} = 10620.84/11 = 965.53 \text{ kN}$$

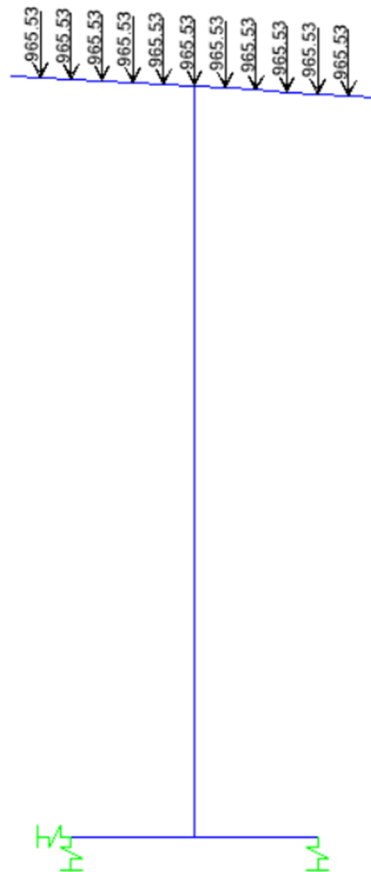


Figure 4.12. Total deck load.

5. ANALYSIS RESULTS

5.1. Modal Analysis results

The fundamental periods and mode shapes of single column pier in transverse direction for flexural stiffness ratios: 0.30-0.60-0.90 are given in Figures below. Due to cap beam is assigned as a lumped mass over the top of the column in horizontal-only earthquake effect whereas, cap beam is added as frame element in case of vertical and horizontal earthquake effects, mode shapes and periods are given separately for both cases.

Table 5.1. Fundamental periods for flexural stiffness ratio 0.30 – 0.60 – 0.90.

	Flexural Stiffness Ratio, k (cap beam / column)		
	k=0.30	k=0.60	k=0.90
Horizontal-only Model	3.015 sec	3.10 sec	3.163 sec
Horizontal & Vertical Model	3.023 sec	3.103 sec	3.172 sec

5.1.1. Modal Shapes and Periods for Lumped Mass Analysis Model

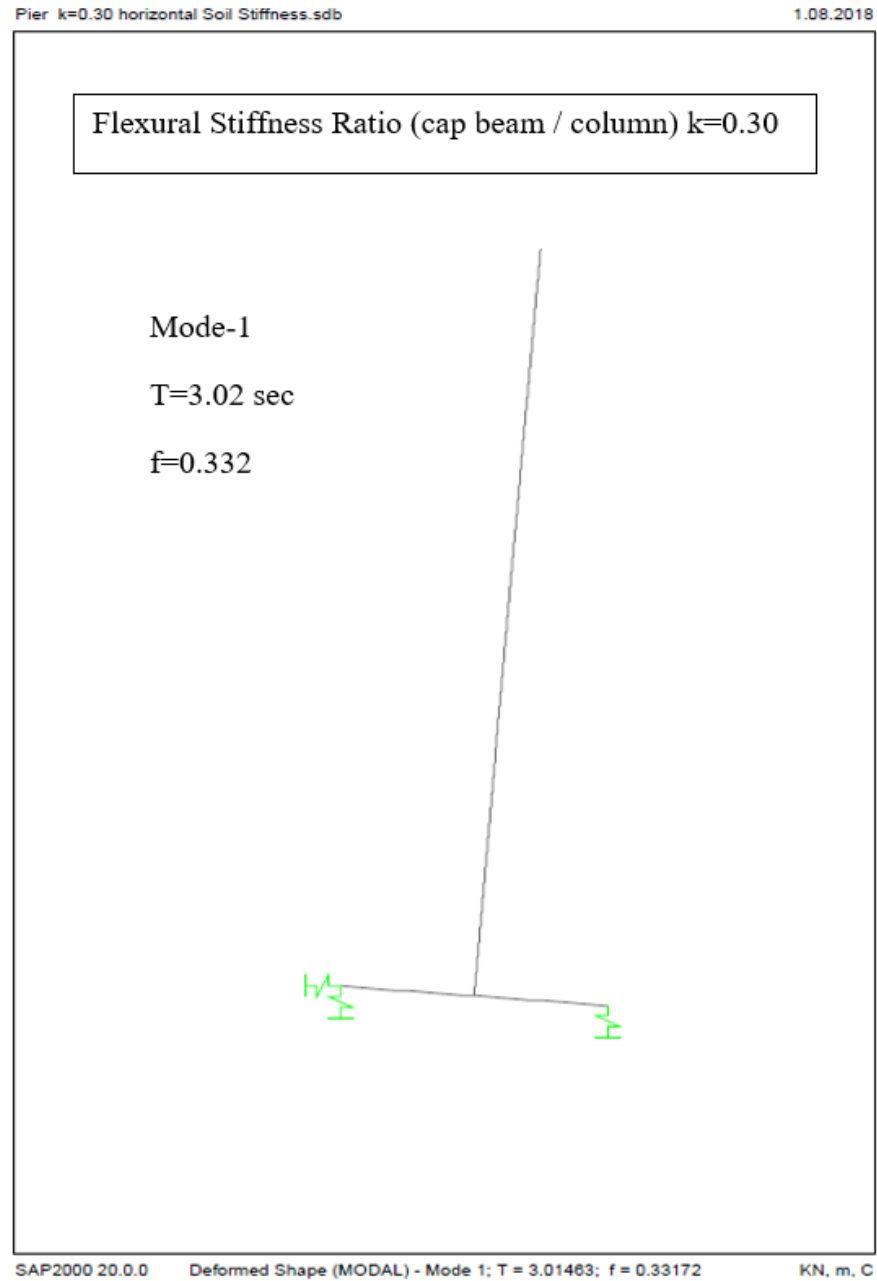


Figure 5.1. Modal shape for flexural stiffness ratio (cap beam/column): 0.30.

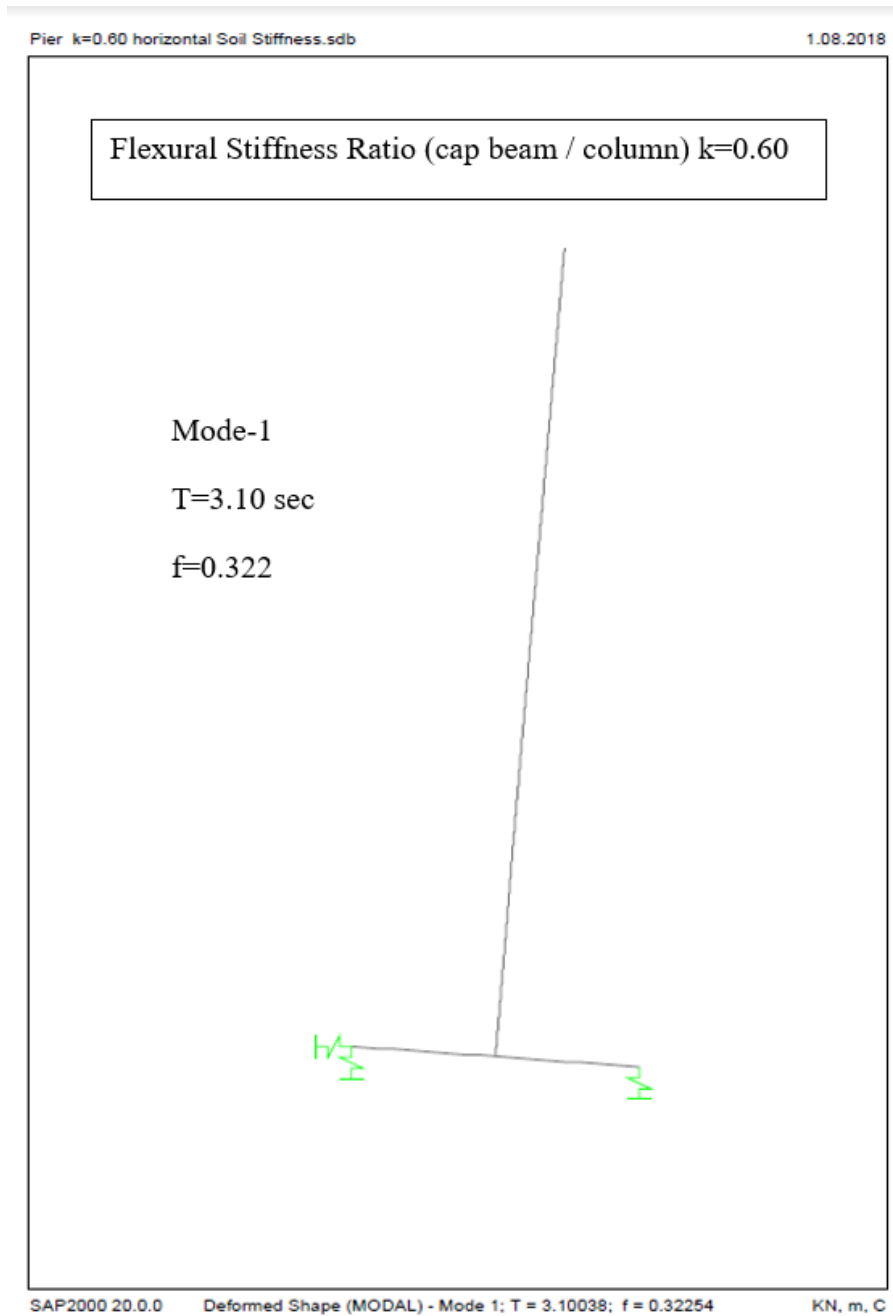


Figure 5.2. Modal shape for flexural stiffness ratio (cap beam/ column): 0.60.

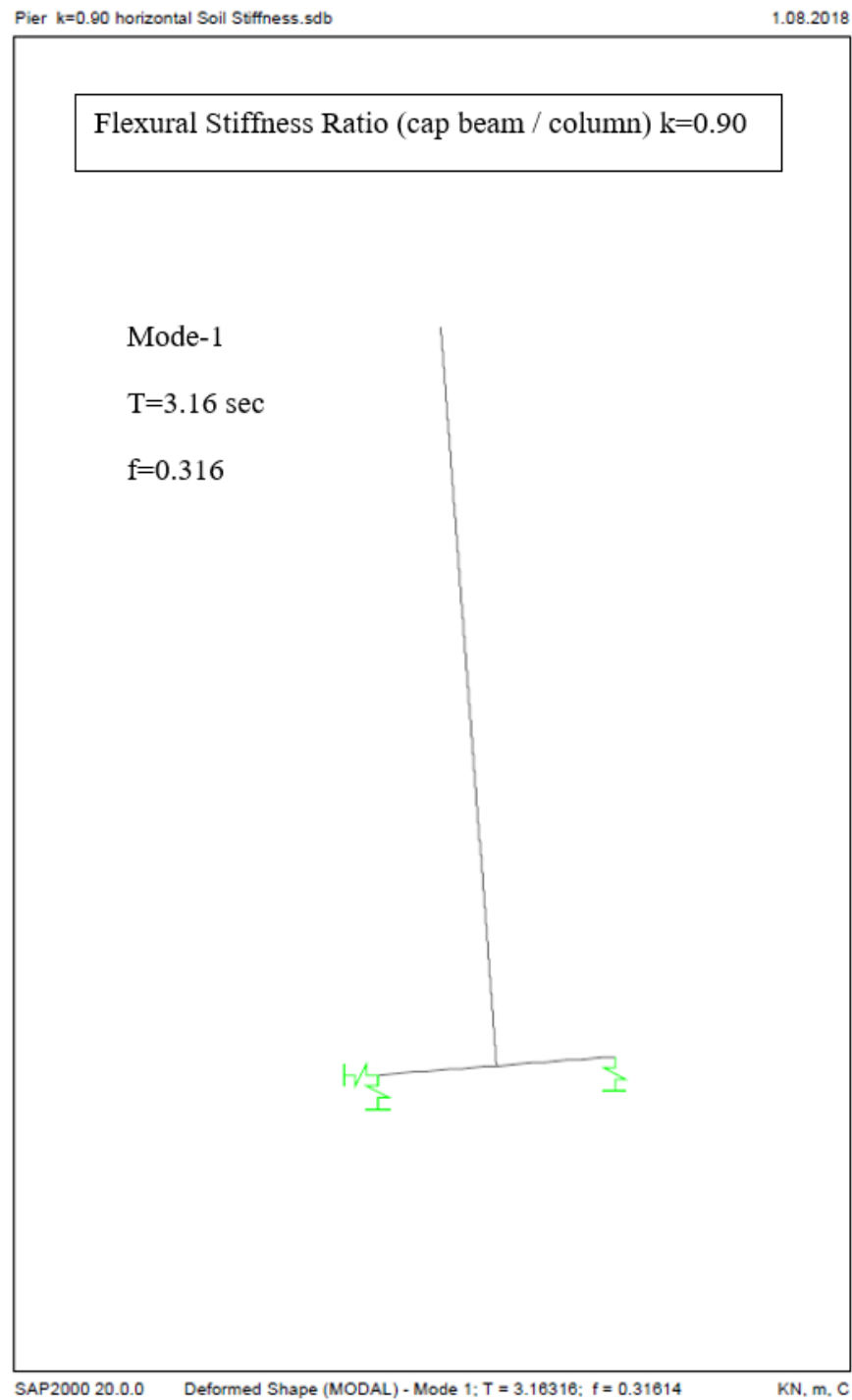


Figure 5.3. Modal shape for flexural stiffness ratio (cap beam/ column): 0.90.

5.1.2. Modal Shapes and Periods for Distributed Mass Analysis Model

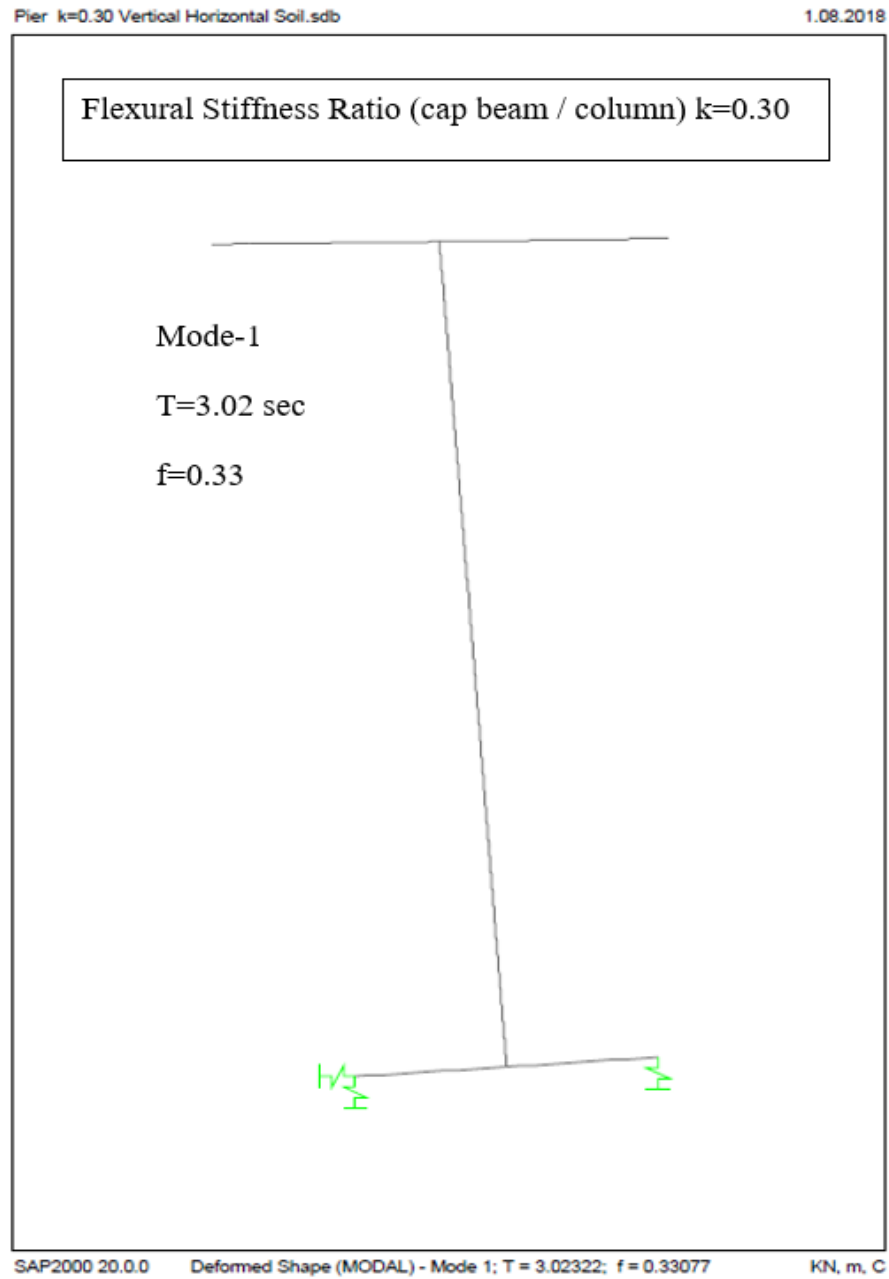


Figure 5.4. Modal shape for flexural stiffness ratio (cap beam/column): 0.30.

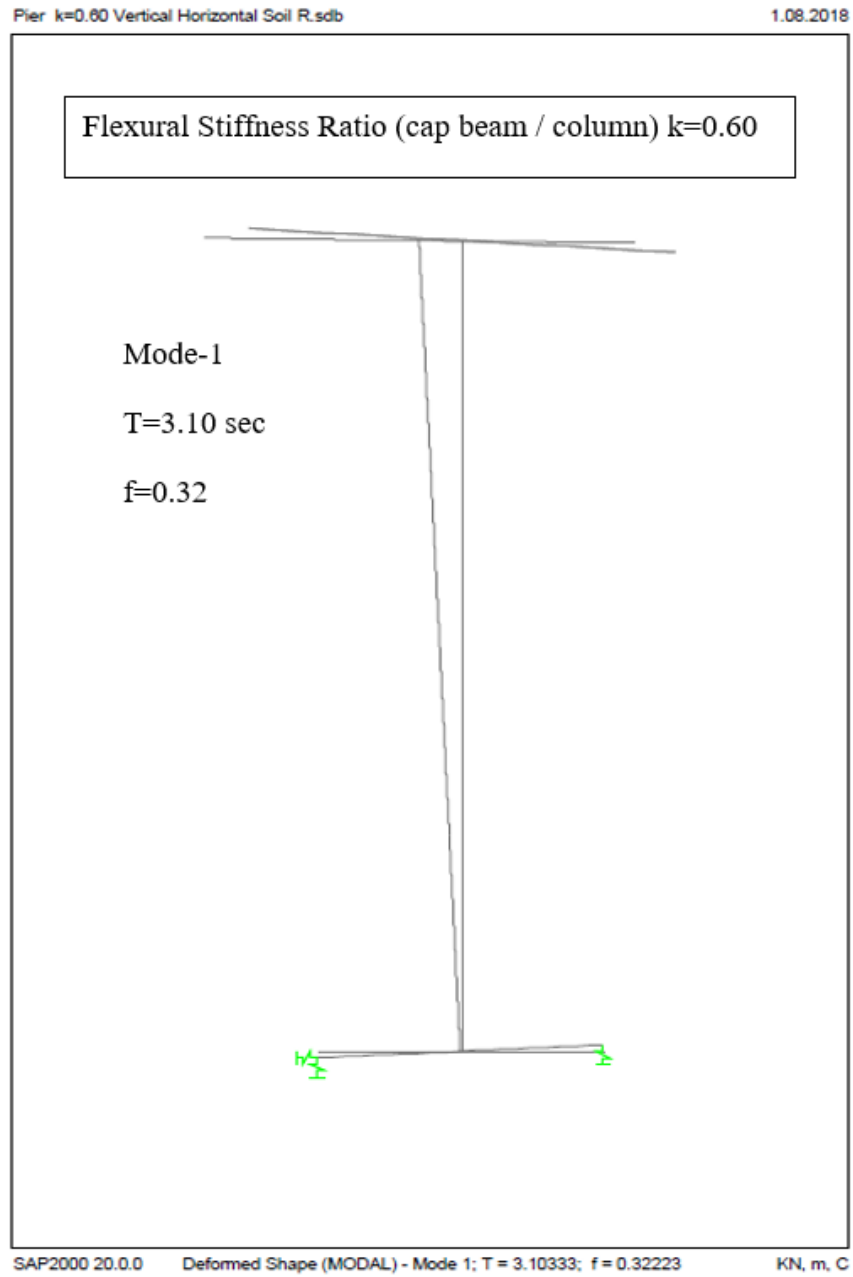


Figure 5.5. Modal shape for flexural stiffness ratio (cap beam/column):0.60.

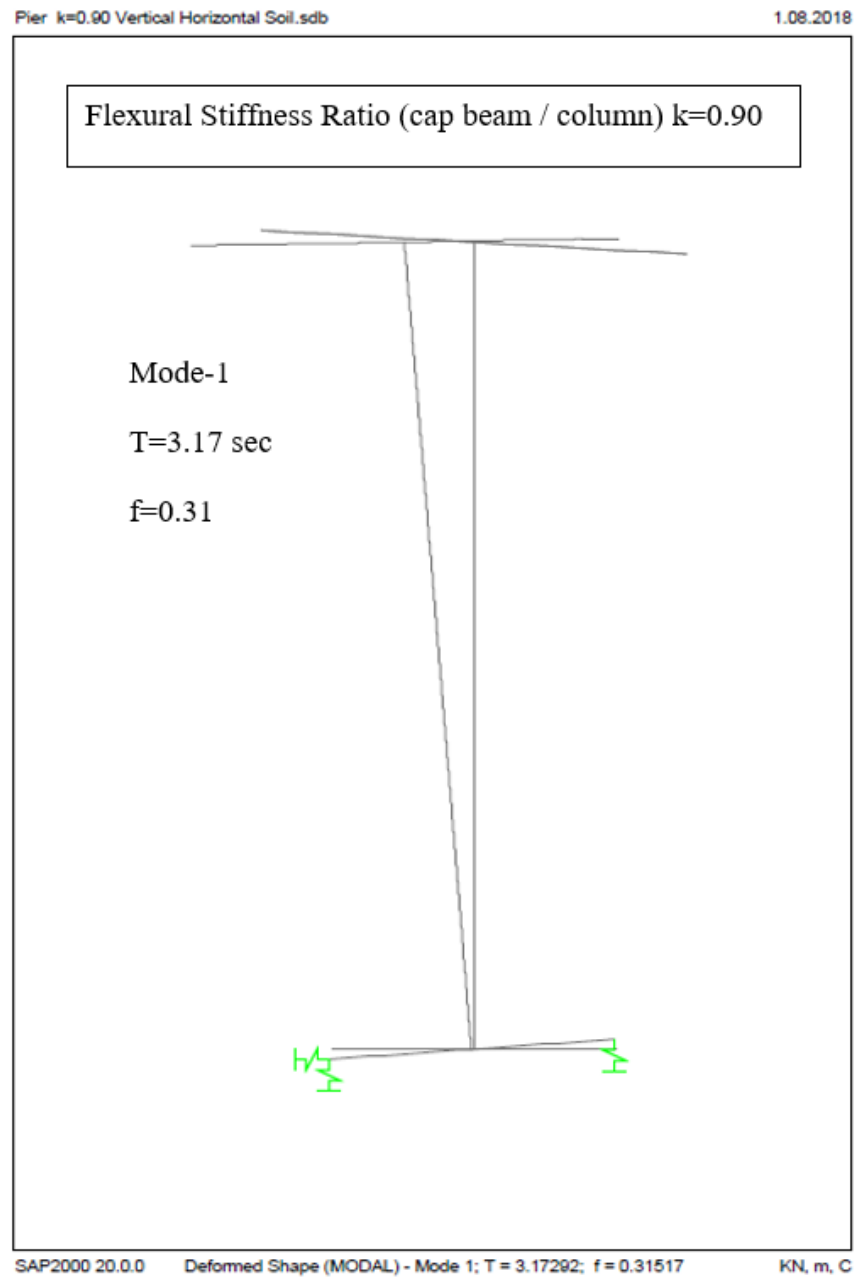


Figure 5.6. Modal shape for flexural Stiffness Ratio (cap beam/column): 0.90.

5.2. Spectral Accelerations for Vertical and Horizontal Component of Ground Motions Records

Elastic acceleration response spectra of horizontal and vertical ground motions are generated from Kobe (1995-station: Port Island), Kocaeli (1999-station: Sakarya), Kocaeli (1999-station: Yarimca) records using Seismosignal software. Fundamental period of the pier at horizontal direction is approximately 3.0 second for flexural stiffness ratios 0.30 – 0.60 – 0.90. On the other hand, the fundamental period of the pier at vertical direction is nearly 0.69 second for the flexural stiffness ratios of 0.3 -0.60 – 0.90. Spectral acceleration values for vertical and horizontal fundamental modal periods are shown on the elastic acceleration response spectra as seen in Figure 5.7., Figure 5.8. and Figure 5.9. respectively.

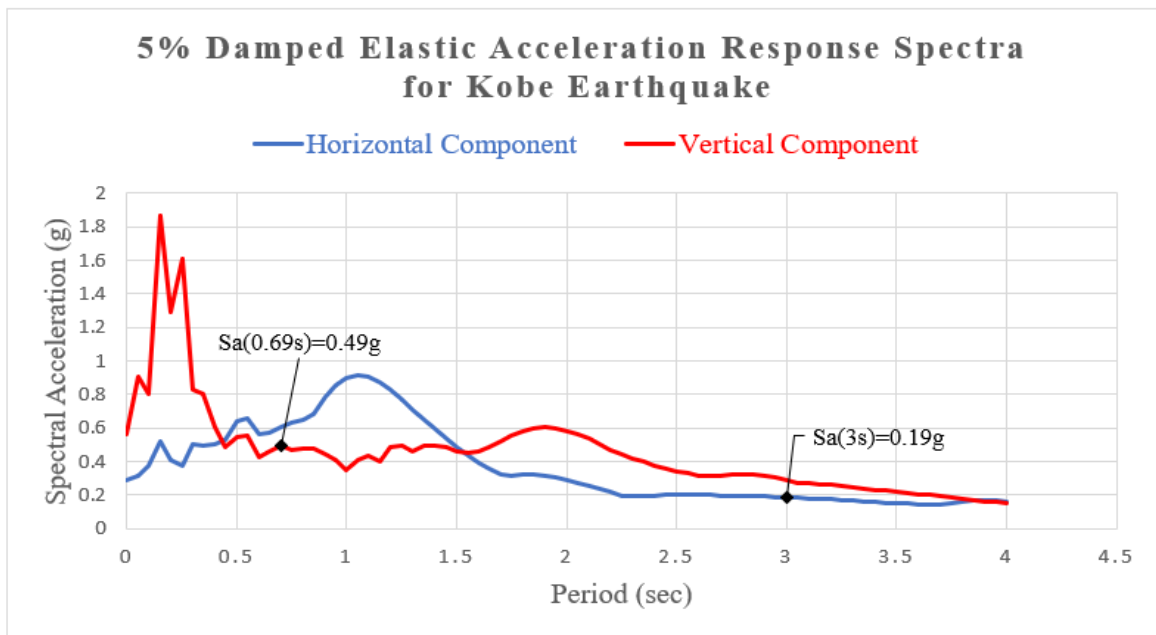


Figure 5.7. 5% damped elastic acceleration response spectra for Kobe earthquake vertical and horizontal components, the dots represent the spectral acceleration values for horizontal and vertical fundamental modal periods.

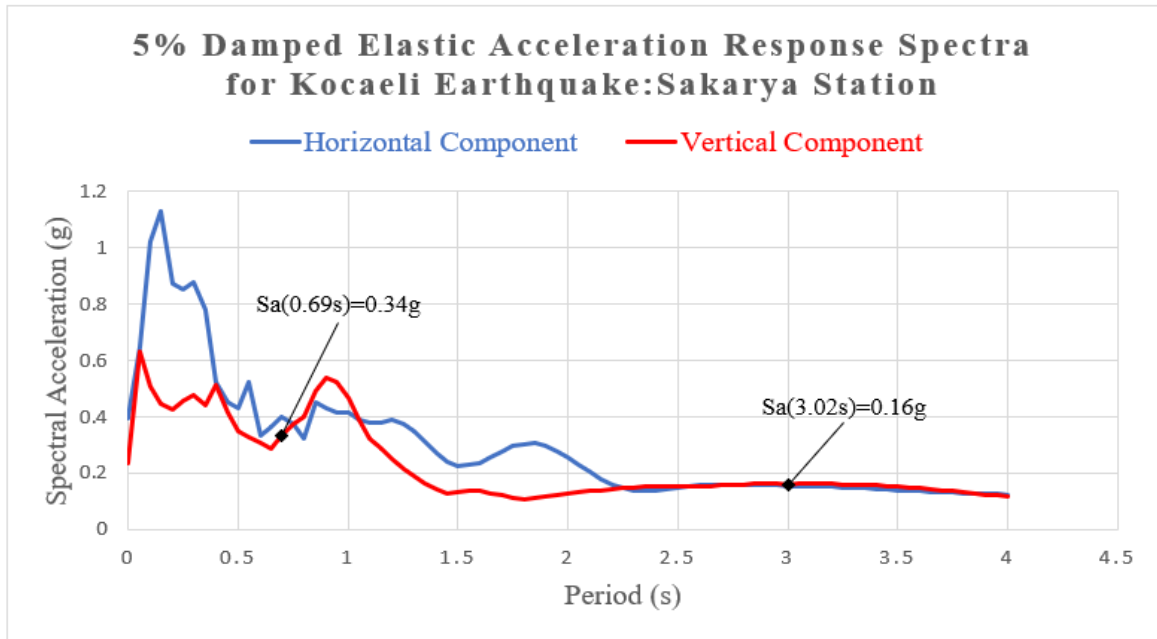


Figure 5.8. 5% damped elastic acceleration response spectra for Sakarya station Kocaeli earthquake vertical and horizontal components, the dots represent the spectral acceleration values for horizontal and vertical fundamental modal periods.

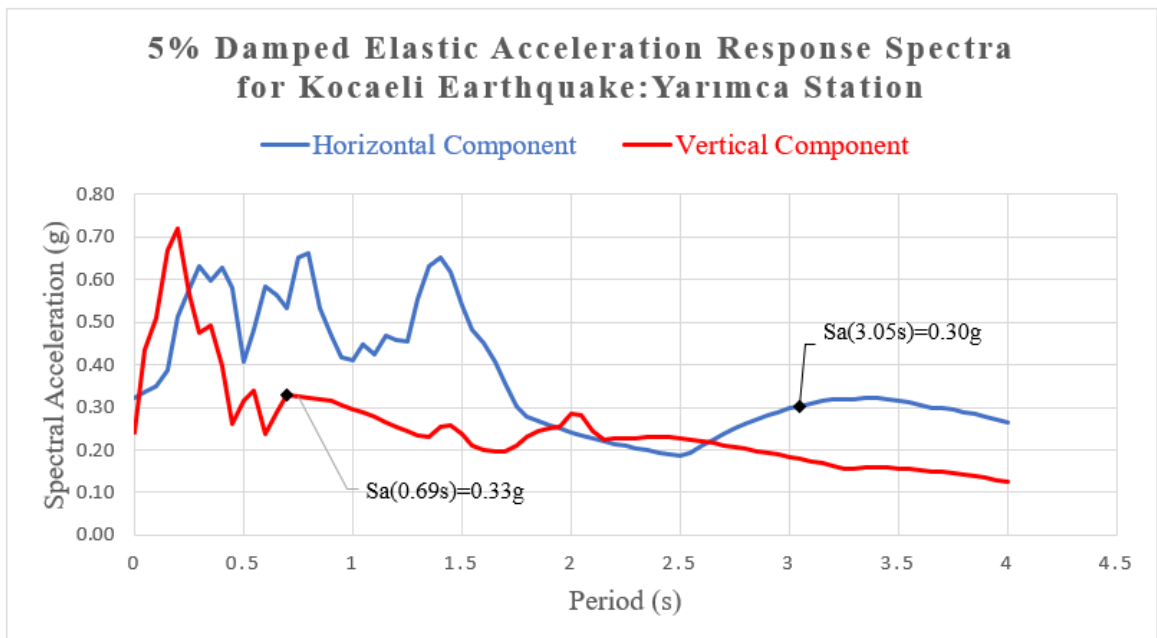


Figure 5.9. 5% damped elastic acceleration response spectra for Yarımca station Kocaeli earthquake vertical and horizontal components, the dots represent the spectral acceleration values for horizontal and vertical fundamental modal periods.

5.3. Time History Analysis Results under Horizontal-only Ground Motion and Both Horizontal and Vertical Ground Motion

Column base shear forces increase 7.5 %, 5.8 % and 1.6 % for flexural stiffness ratios of 0.30 – 0.60 - 0.90 respectively under the horizontal and vertical component of Kobe earthquake-Port Island station with respect to horizontal-only ground motion

Column base shear forces increase 4.5 %, 5.6 % and 6.6 % for flexural stiffness ratios of 0.30 – 0.60 - 0.90 respectively under the horizontal and vertical component of Kocaeli earthquake-Sakarya station with respect to horizontal-only ground motion.

Under the horizontal and vertical component of Kocaeli earthquake-Yarimca station, column base shear forces increase 2.2 %, 2.8 % and 8.9 % for flexural stiffness ratios of 0.30 – 0.60 - 0.90 with respect to horizontal-only ground motion.

Table 5.2. Shear forces (kN) under horizontal and both vertical and horizontal ground motion for flexural stiffness ratios: 0.30 – 0.60 – 0.90.

Flexural stiffness ratio	KOBE			SAKARYA			YARIMCA		
	Horizontal Only	Horizontal +Vertical	Increase %	Horizontal Only	Horizontal +Vertical	Increase %	Horizontal Only	Horizontal +Vertical	Increase %
0.3	3302	3430	3.9%	3095	3236	4.6%	6460	6646	2.9%
0.6	3356	3495	4.1%	3133	3309	5.6%	7140	7298	2.2%
0.9	3360	3534	5.2%	3120	3327	6.6%	7060	7668	8.6%

The horizontal and vertical component of ground motion effect did not create any increase of bending moment at the base of the column with respect to horizontal only ground motion.

Table 5.3. Moment diagrams (kNm) under horizontal-only and both horizontal and vertical earthquake for flexural stiffness ratios 0.30 – 0.60 – 0.90.

Flexural stiffness ratio	KOBE		SAKARYA		YARIMCA	
	Horizontal Only	Horizontal +Vertical	Horizontal Only	Horizontal +Vertical	Horizontal Only	Horizontal +Vertical
0.3	104811	102278	87526	85493	197260	188389
0.6	106709	104431	89966	87621	207613	201113
0.9	107452	105746	91548	89498	213611	206901

5.3.1. Column Shear Force Diagrams for Flexural Stiffness Ratio: 0.30

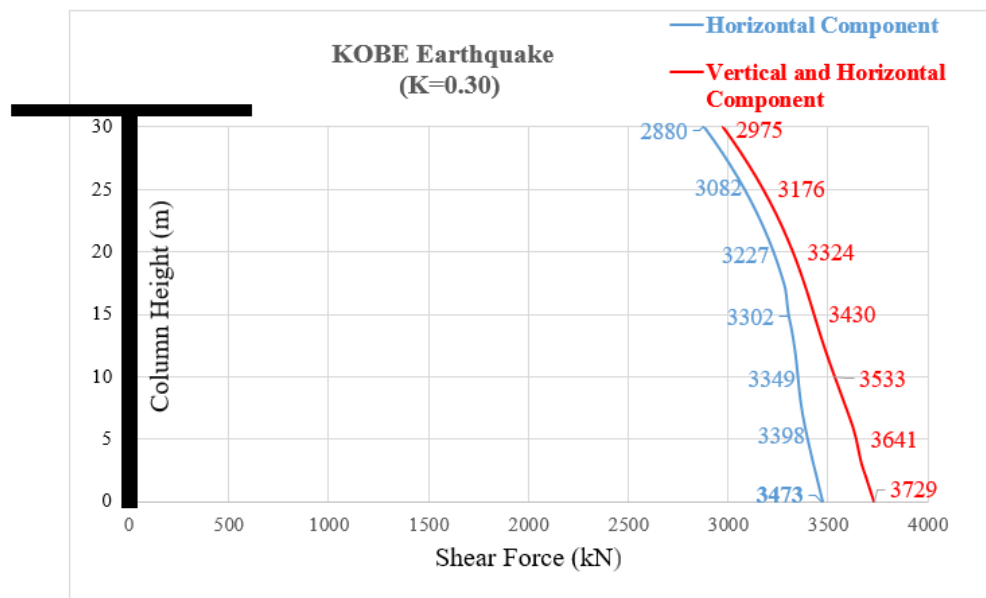


Figure 5.10. Shear force diagrams: Kobe earthquake flexural stiffness ratio: 0.30.

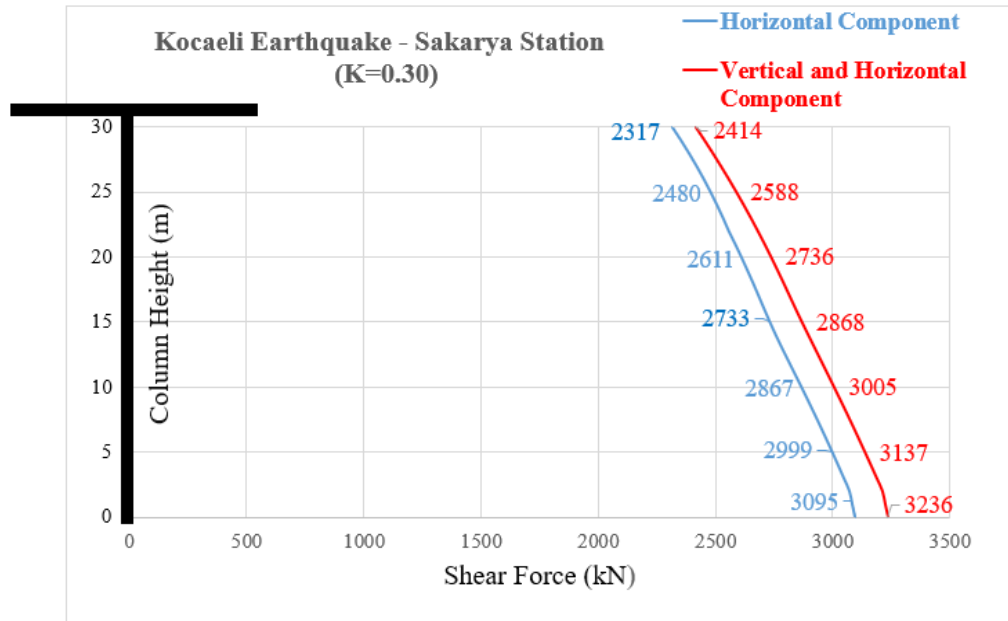


Figure 5.11. Shear force diagrams: Kocaeli earthquake Sakarya station flexural stiffness ratio: 0.30.

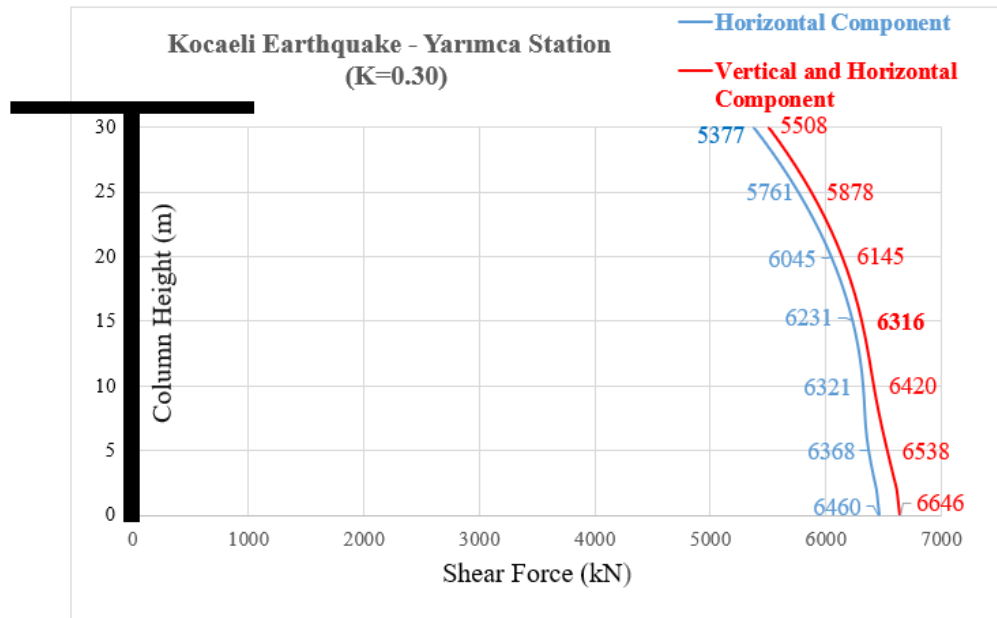


Figure 5.12. Shear force diagrams: Kocaeli earthquake Yarimca station flexural stiffness ratio: 0.30.

5.3.2. Column Moment Diagrams for Flexural Stiffness Ratio: 0.30

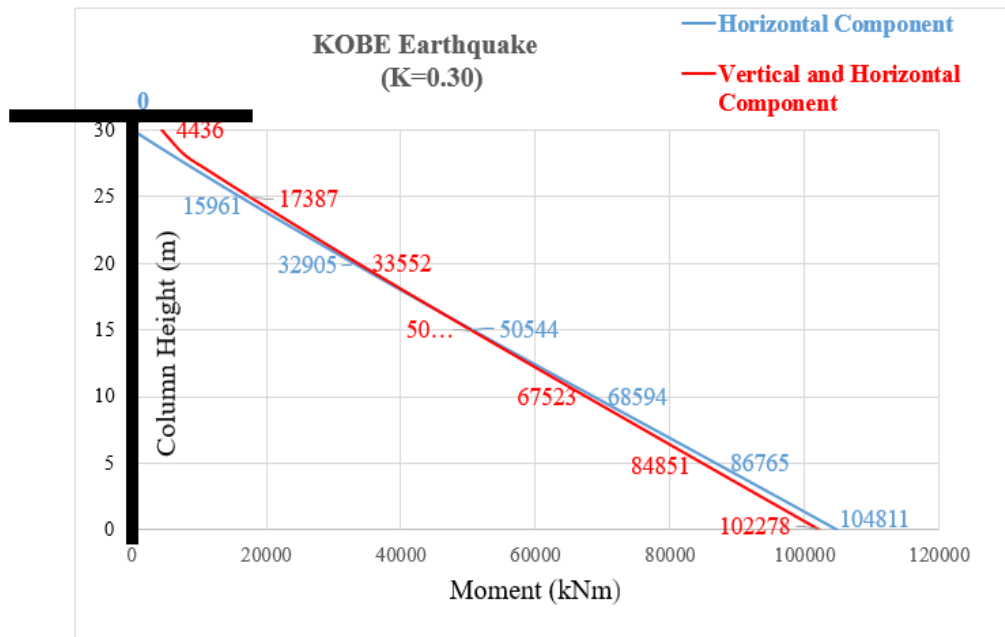


Figure 5.13. Moment diagrams: Kobe earthquake flexural stiffness ratio: 0.30.

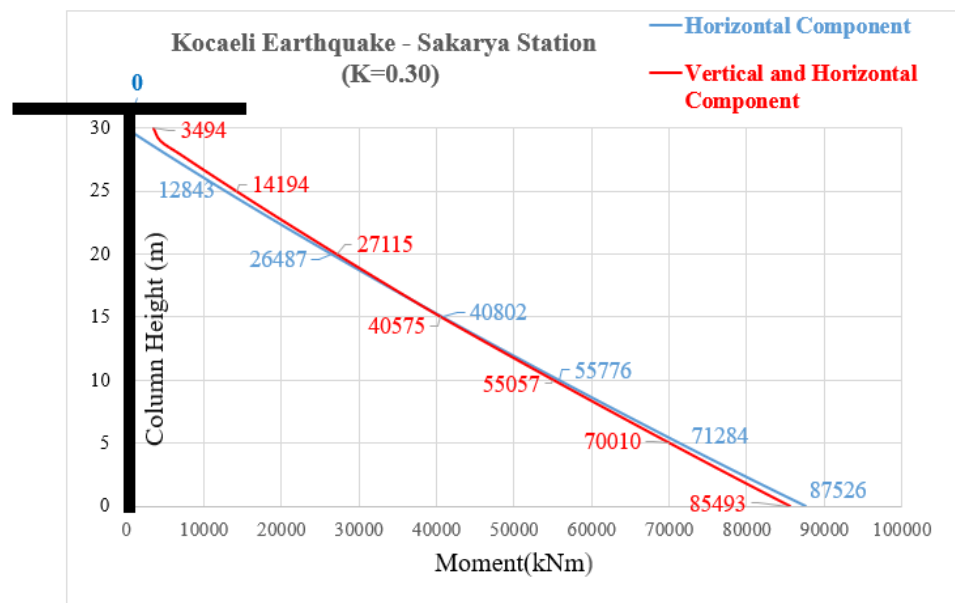


Figure 5.14. Moment diagrams: Kocaeli earthquake-Sakarya station flexural stiffness ratio: 0.30.

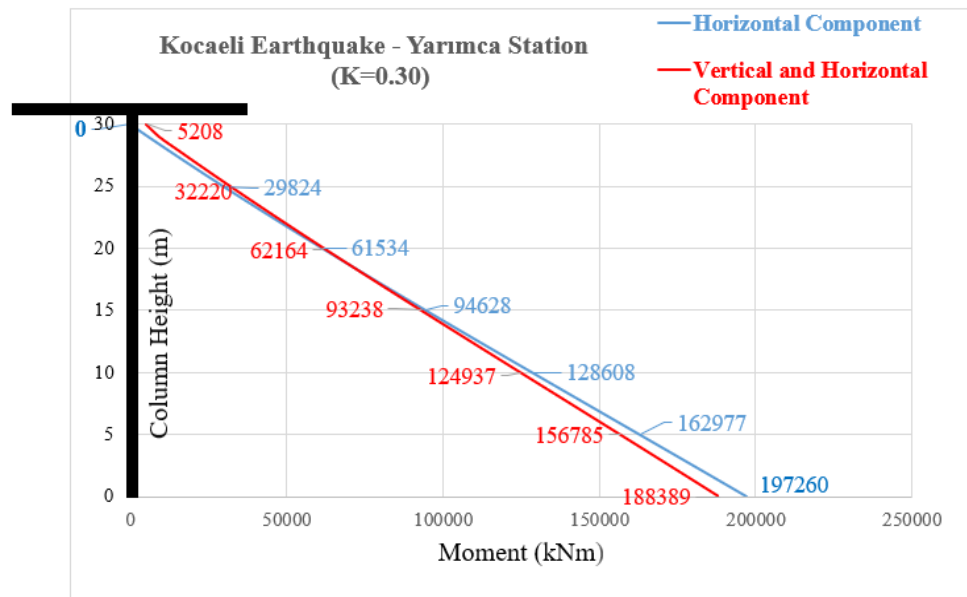


Figure 5.15. Moment diagrams: Kocaeli earthquake-Yarimca station flexural stiffness ratio: 0.30.

5.3.3. Column Shear Force Diagrams for Flexural Stiffness Ratio: 0.60

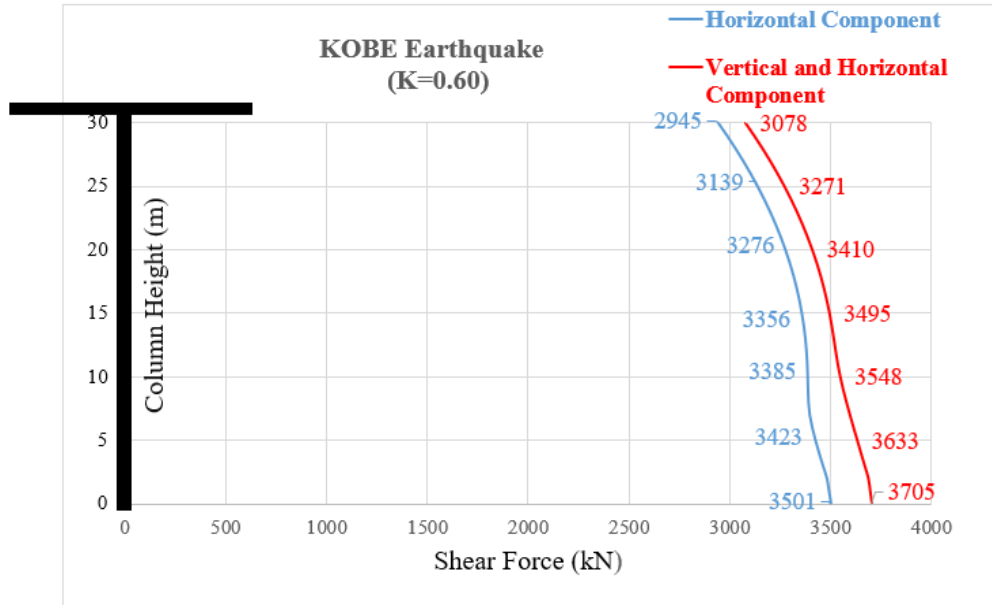


Figure 5.16. Shear force diagrams: Kobe earthquake flexural stiffness ratio: 0.60.

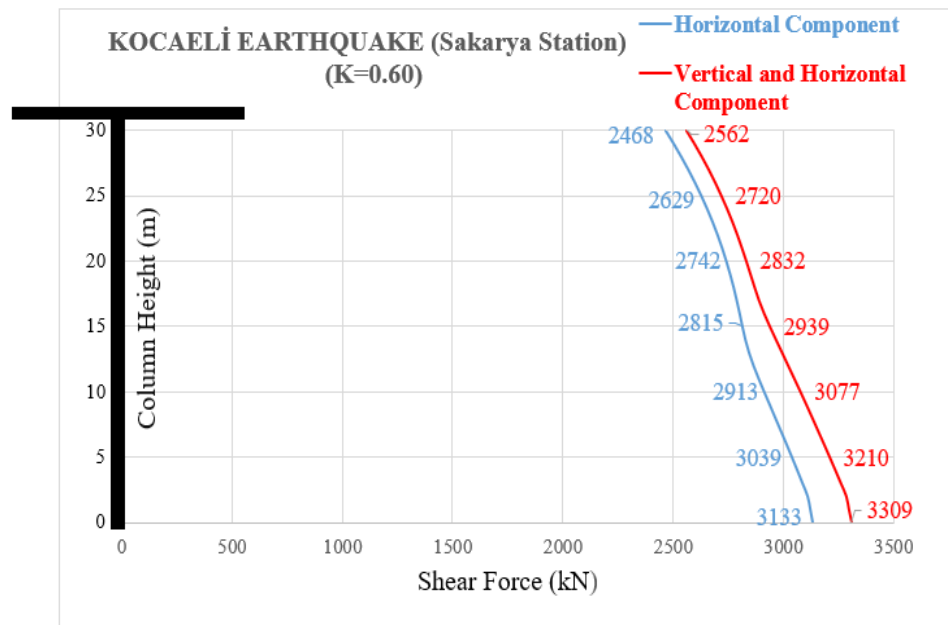


Figure 5.17. Shear force diagrams: Kocaeli earthquake Sakarya station flexural stiffness ratio: 0.60.

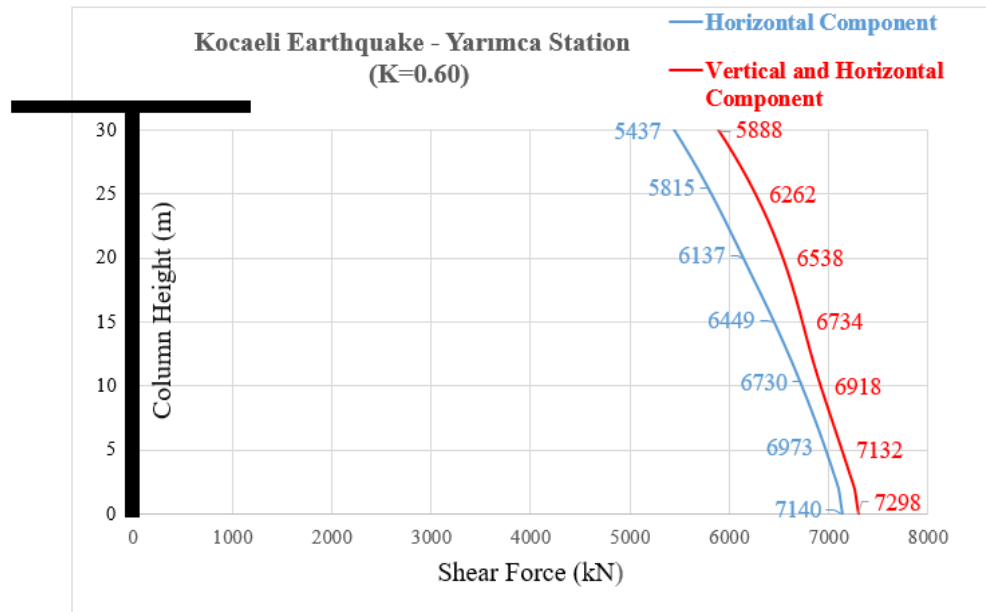


Figure 5.18. Shear force diagrams: Kocaeli earthquake Yarimca station flexural stiffness ratio: 0.60.

5.3.4. Column Moment Diagrams for Flexural Stiffness Ratio: 0.60

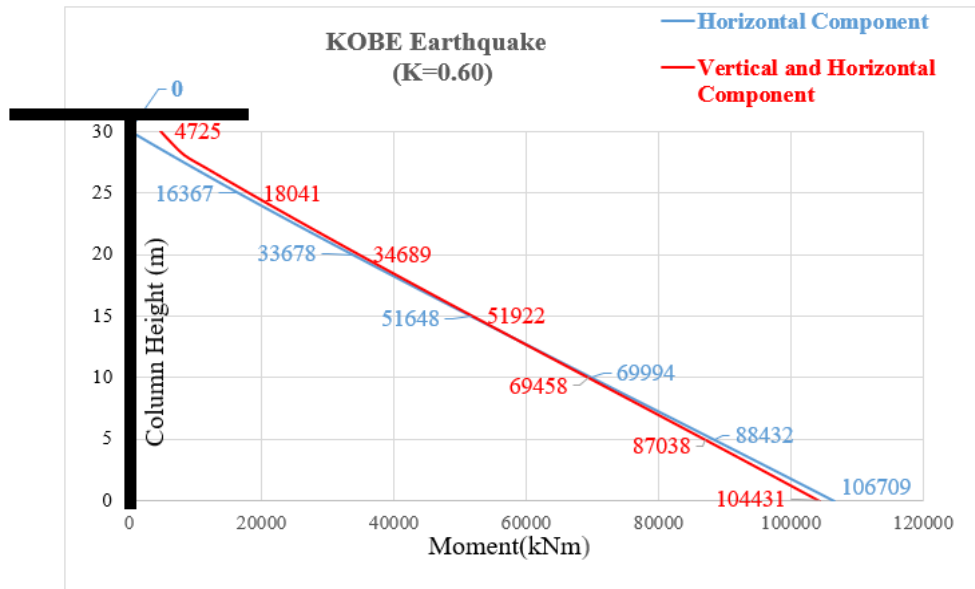


Figure 5.19. Moment diagrams: Kobe earthquake flexural stiffness ratio: 0.60.

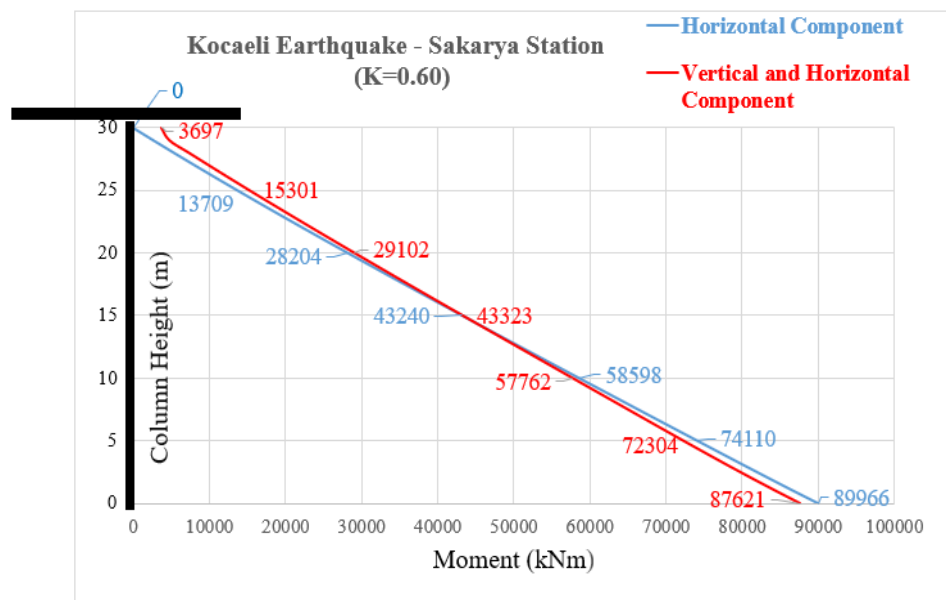


Figure 5.20. Moment diagrams: Kocaeli earthquake-Sakarya station flexural stiffness ratio: 0.60.

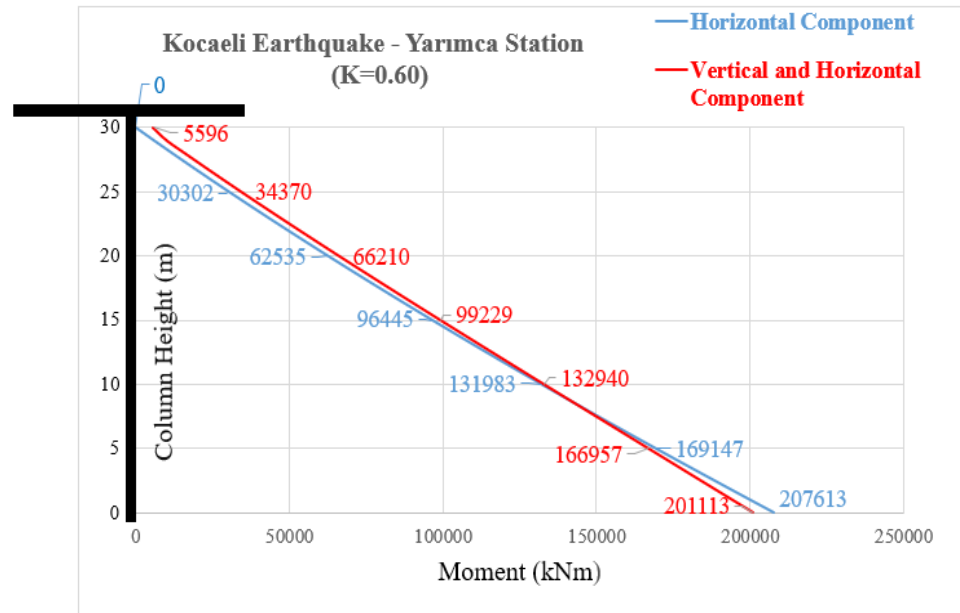


Figure 5.21. Moment diagrams: Kocaeli earthquake-Yarimca station flexural stiffness ratio: 0.60.

5.3.5. Column Shear Force Diagrams for Flexural Stiffness Ratio: 0.90

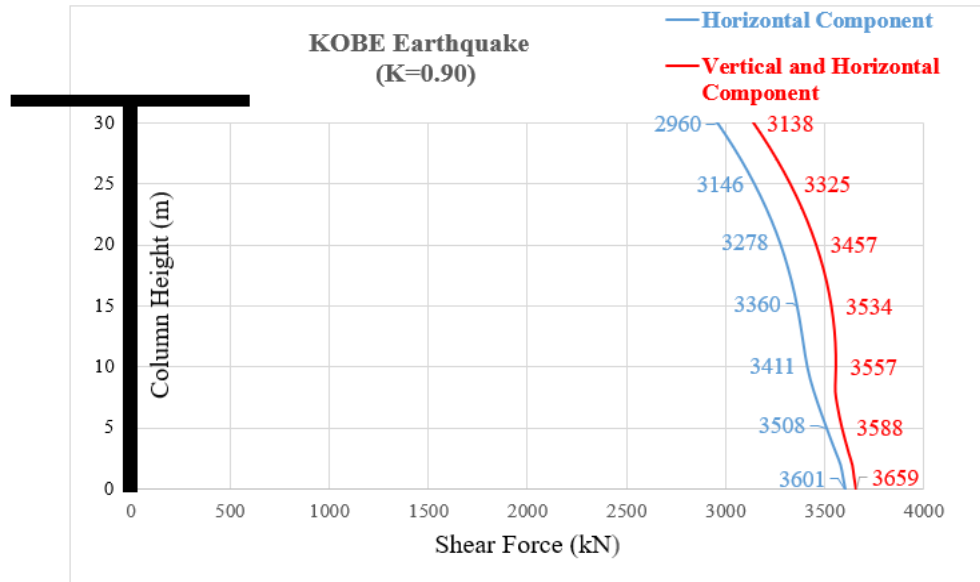


Figure 5.22. Shear force diagrams: Kobe earthquake flexural stiffness ratio: 0.90.

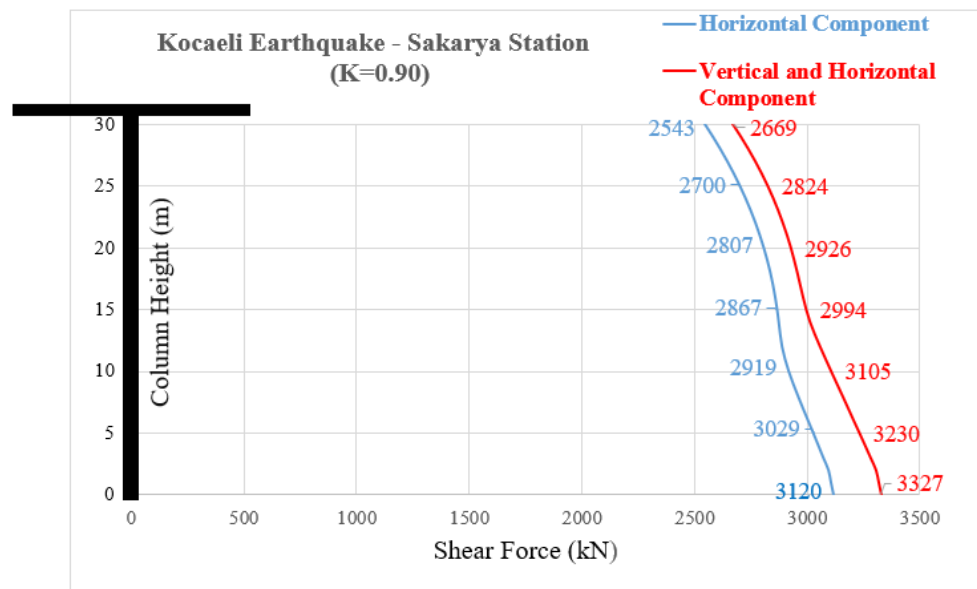


Figure 5.23. Shear force diagrams: Kocaeli earthquake Sakarya station flexural stiffness ratio: 0.90.

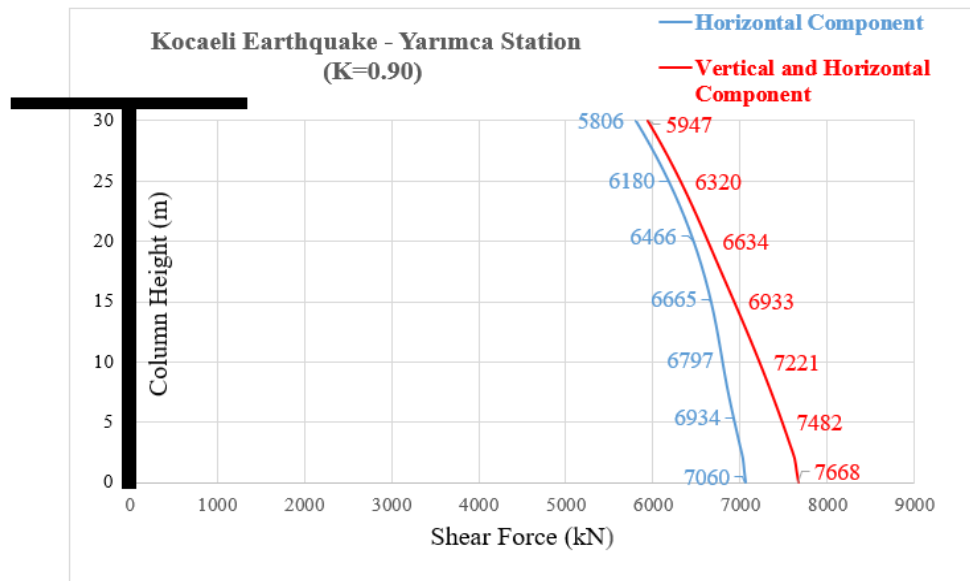


Figure 5.24. Shear force diagrams: Kocaeli earthquake Yarımca station flexural stiffness ratio: 0.90.

5.3.6. Column Moment Diagrams for Flexural Stiffness Ratio: 0.90

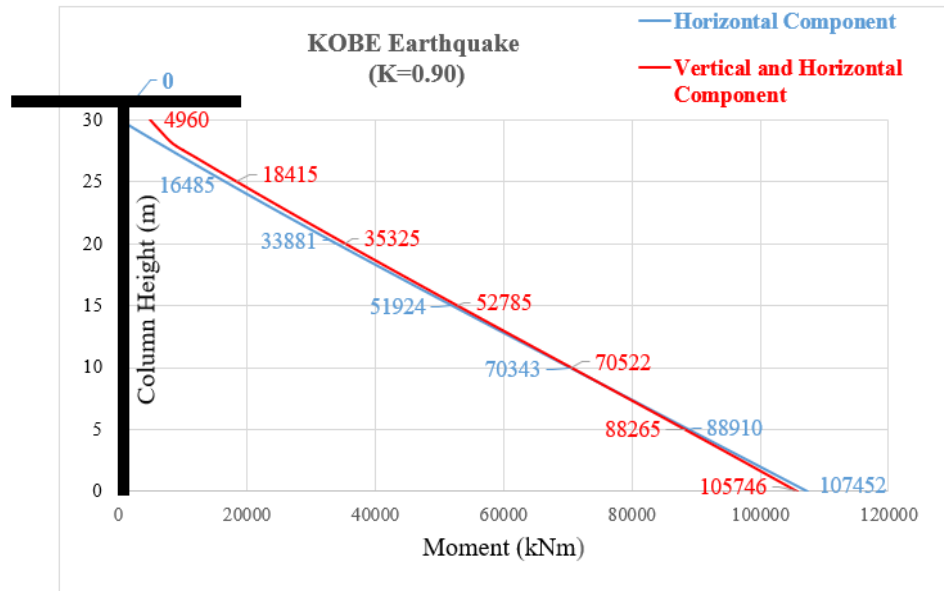


Figure 5.25. Moment diagrams: Kobe earthquake flexural stiffness ratio: 0.90.

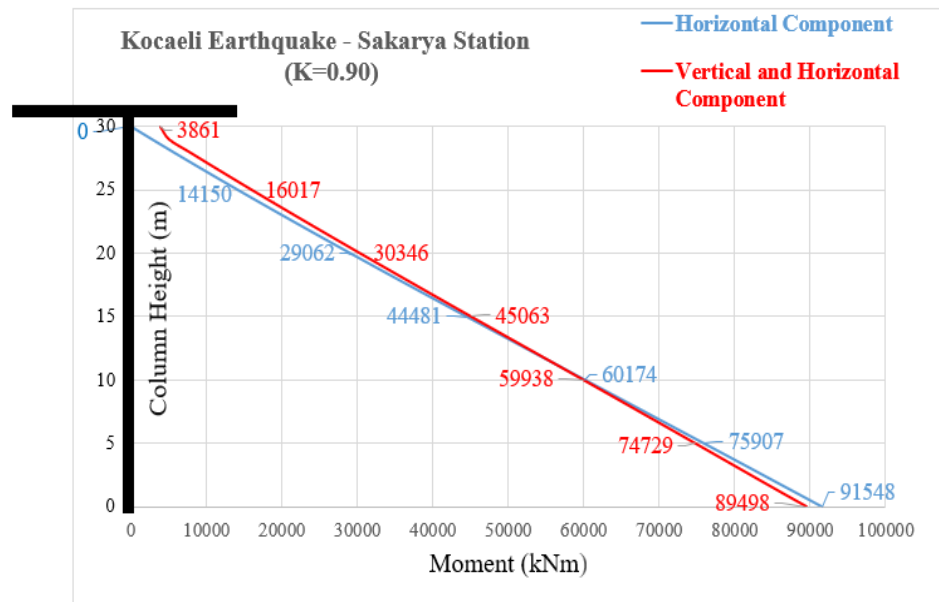


Figure 5.26. Moment diagrams: Kocaeli earthquake-Sakarya station flexural stiffness ratio: 0.90.

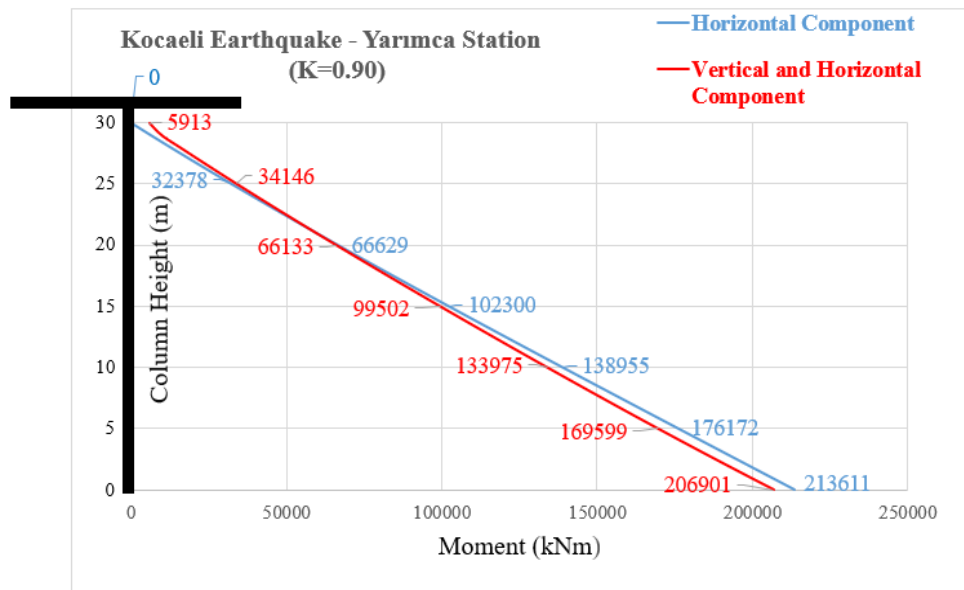


Figure 5.27. Moment diagrams: Kocaeli earthquake-Yarimca station flexural stiffness ratio: 0.90.

6. SEISMIC PERFORMANCE OF SINGLE COLUMN BRIDGE PIER UNDER DIFFERENT DEPTH OF SOIL EMBEDMENT

6.1. P-y Curves for Lateral Soil Resistance-Deflection

The lateral soil resistance-deflection is represented by p-y curves according to guide of American Petroleum Institute (API 2A,2000) and Reese and Van Impe (2001). P-y relationship is nonlinear for clay and granular soils. Generally granular soil is used as filler material for a dump / storage area. Therefore, load deflection curves for sand has been conducted. Angle of friction of sand is taken as, $\phi = 35^\circ$ in the analysis. It is approximated at any specific depth H, by the following expression:

$$p = A \times p_u \times \tanh \left[\frac{k \times H}{A \times p_u} \times y \right] \quad (6.1.)$$

Where:

$A = 0.9$ for cycling loading

$= (3.0 - 0.8 \times H/D) \geq 0.9$ for static loading

p_u : Ultimate bearing capacity at depth H (kN/m)

k : Initial modulus of subgrade reaction (kN/m³)

y : Lateral deflection (m)

H : Depth (m)

The ultimate lateral bearing capacity p_u changes through depth of sand layer. The value of p_u is calculated by the following equations. Smaller of p_u values from below should be used.

$$p_{us} = (C_1 \times H + C_2 \times D) \times \gamma \times H \quad (6.2)$$

$$p_{ud} = C_3 \times D \times \gamma \times H \quad (6.3)$$

p_u : Ultimate resistance (kN/m) (s: shallow, d: deep)

γ : Effective soil weight (kN/m³)

H : Depth (m)

ϕ' : Angle of internal friction of sand (deg)

C_1, C_2, C_3 : Coefficients

D : Average column diameter in transverse direction (m)

C_1, C_2, C_3 coefficients according to angle of friction of sand, $\phi' = 35$ is obtained from the Figure 6.1. given below.

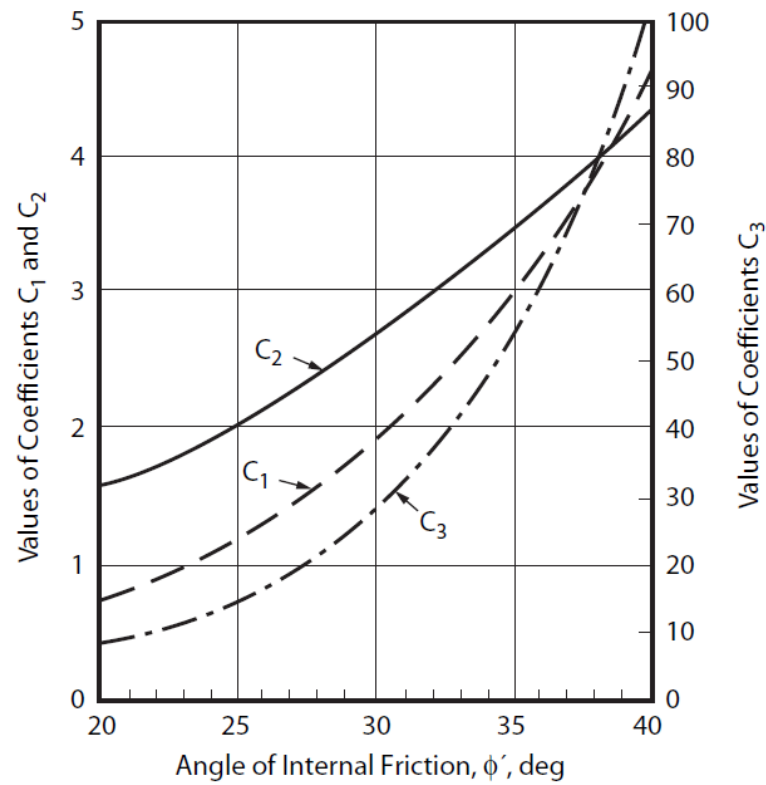


Figure 6.1. C_1, C_2, C_3 Coefficients according to angle of internal friction (API 2A,2000).

Modulus of subgrade reaction according to angle of friction of sand, $\phi = 35^\circ$ is obtained from the Figure 6.2. given below. Sand is considered above water table.

$\phi' = 35^\circ \rightarrow k = 150 \left(\frac{\text{lb}}{\text{in}^3} \right) = 38921 \text{ kN/m}^3$ (Modulus of subgrade reaction k for $\phi = 35^\circ$)

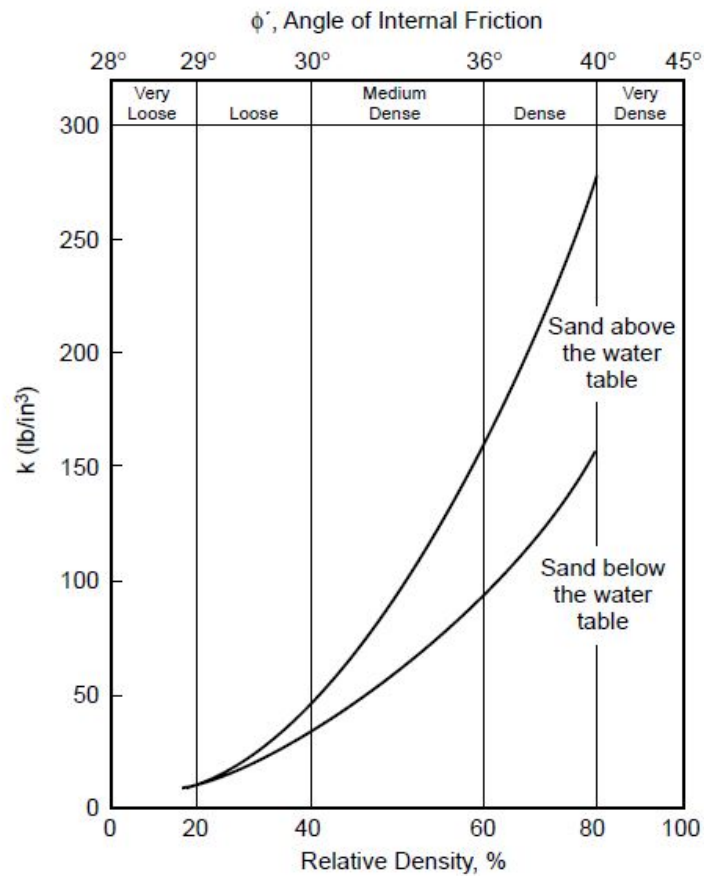


Figure 6.2. Modulus of subgrade reaction according to angle of internal friction (API 2A,2000).

P-y relationship is constructed for each 1m soil depth and assigned as multilinear plastic link property at Sap 2000 as seen in below. Force deformation definition and Hysteresis type used is shown below in Figure 6.4. Only compression should be specified for the soil with gapping behavior therefore multilinear link elements are assigned to both sides of the column. Tensile stiffness is set to zero and hysteretic model is selected as Takeda.

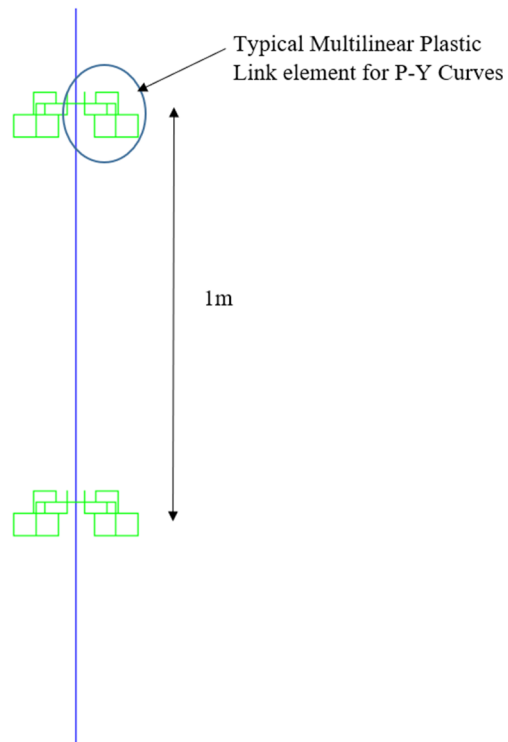


Figure 6.3. Typical multilinear plastic link property defined for p-y curve.

The soil-column hysteretic behavior in the analysis model is simulated using nonlinear link elements with the hysteresis model of Takeda as shown in Figure 6.4. Although the model of Takeda is usually used for concrete members, it is similar to the hysteretic rules suggested by Shirato (2006). Past experiments about piles subjected to lateral loading were examined then analytical study of soil element behavior were undertaken by Shirato. Thus, it is used for the simulation of the hysteretic behavior of the soil- column system.

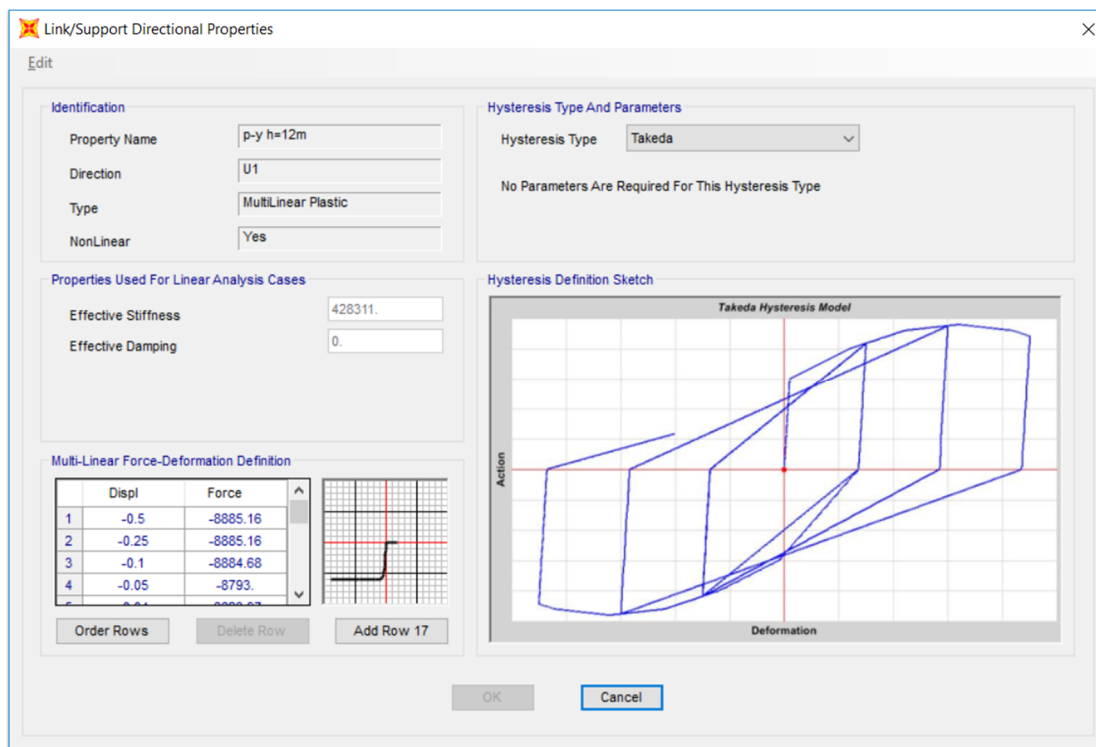


Figure 6.4. Force-deformation definition and takeda hysteresis type (Sap2000 Link/Support Directional Properties).

H(m)	P _{us}	P _{ud}	P _u (kN/m)	y (m)	P
1	216.60	2612.50	216.60	0.000	0.00
				0.002	73.95
				0.004	129.30
				0.006	162.39
				0.008	179.59
				0.010	187.88
				0.015	193.97
				0.020	194.81
				0.025	194.92
				0.030	194.94
				0.040	194.94
				0.050	194.94
				0.100	194.94
				0.250	194.94
				0.500	194.94

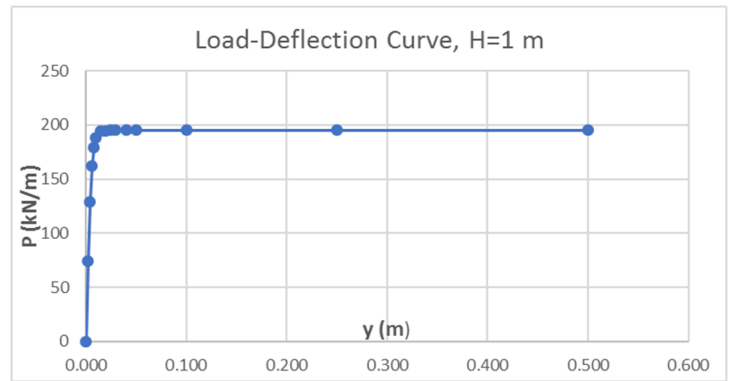


Figure 6.5. Typical multilinear plastic link property defined for p-y curve, h=1 m.

H(m)	P _{us}	P _{ud}	P _u (kN/m)	y (m)	P
2	543.40	5225.00	543.40	0.000	0.00
				0.002	150.63
				0.004	275.16
				0.006	362.90
				0.008	418.00
				0.010	450.13
				0.015	480.88
				0.020	487.38
				0.025	488.72
				0.030	488.99
				0.040	489.06
				0.050	489.06
				0.100	489.06
				0.250	489.06
				0.500	489.06

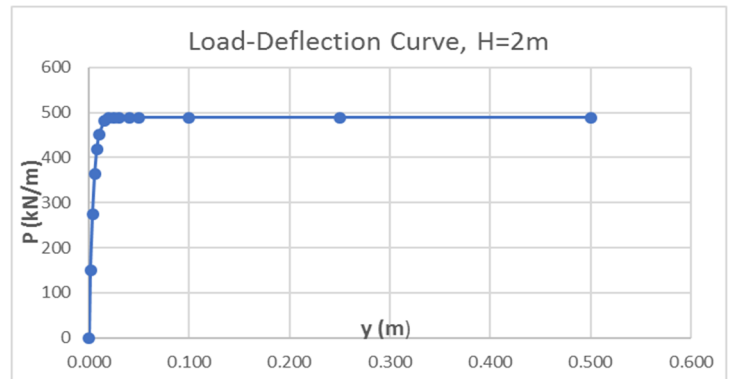


Figure 6.6. Typical multilinear plastic link property defined for p-y curve, h=2 m.

H(m)	P _{us}	P _{ud}	P _u (kN/m)	y (m)	P
3	980.40	7837.50	980.40	0.000	0.00
				0.002	228.22
				0.004	427.82
				0.006	582.94
				0.008	692.78
				0.010	765.54
				0.015	849.67
				0.020	873.54
				0.025	880.00
				0.030	881.73
				0.040	882.32
				0.050	882.36
				0.100	882.36
				0.250	882.36
				0.500	882.36

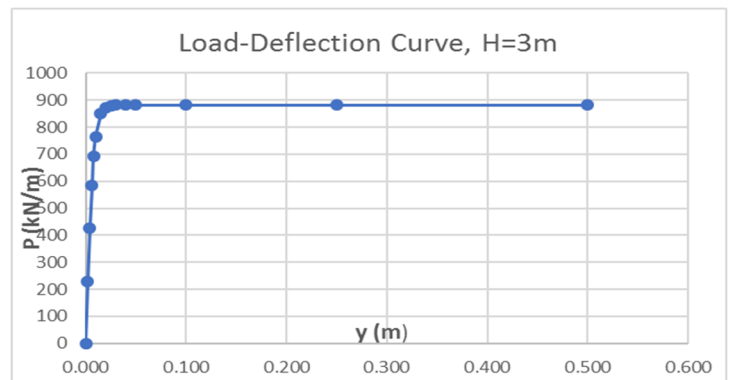


Figure 6.7. Typical multilinear plastic link property defined for p-y curve, h=3 m.

H(m)	P _{us}	P _{ud}	Pu (kN/m)	y (m)	P
4	1527.60	10450.00	1527.60	0.000	0.00
				0.002	306.15
				0.004	583.38
				0.006	812.73
				0.008	988.73
				0.010	1116.14
				0.015	1285.79
				0.020	1345.50
				0.025	1365.32
				0.030	1371.76
				0.040	1374.52
				0.050	1374.81
				0.100	1374.84
				0.250	1374.84
				0.500	1374.84

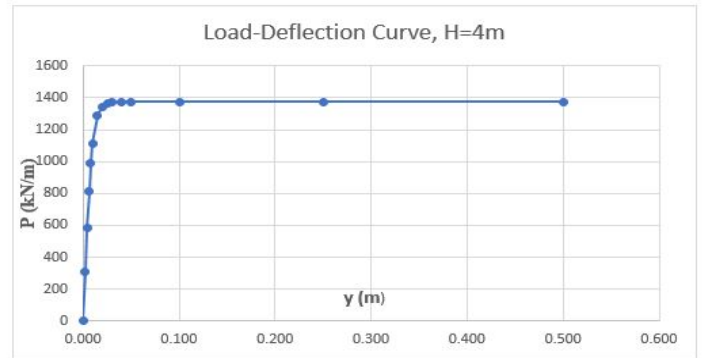


Figure 6.8. Typical multilinear plastic link property defined for p-y curve, h=4 m.

H(m)	P _{us}	P _{ud}	Pu (kN/m)	y (m)	P
5	2185.00	13062.50	2185.00	0.000	0.00
				0.002	384.21
				0.004	740.16
				0.006	1047.35
				0.008	1296.63
				0.010	1489.02
				0.015	1774.35
				0.020	1892.81
				0.025	1938.78
				0.030	1956.15
				0.040	1965.07
				0.050	1966.30
				0.100	1966.50
				0.250	1966.50
				0.500	1966.50

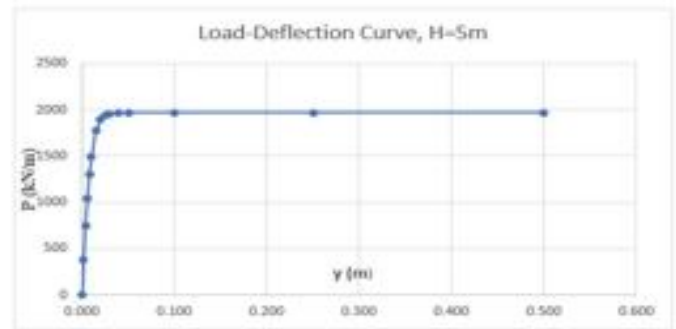


Figure 6.9. Typical multilinear plastic link property defined for p-y curve, h=5 m.

H(m)	P _{us}	P _{ud}	Pu (kN/m)	y (m)	P
6	2952.60	15675.00	2952.60	0.000	0.00
				0.002	462.30
				0.004	897.44
				0.006	1284.29
				0.008	1611.12
				0.010	1875.59
				0.015	2302.14
				0.020	2503.83
				0.025	2592.50
				0.030	2630.22
				0.040	2652.64
				0.050	2656.53
				0.100	2657.34
				0.250	2657.34
				0.500	2657.34

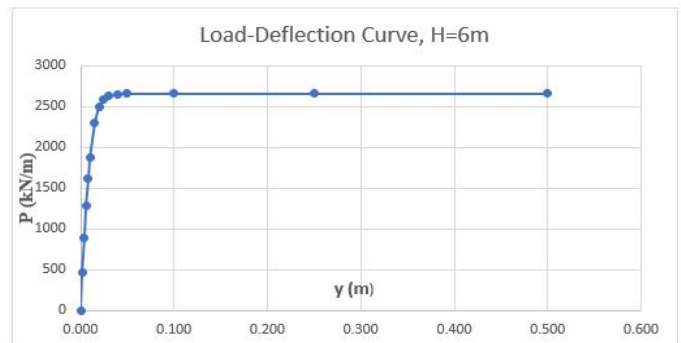


Figure 6.10. Typical multilinear plastic link property defined for p-y curve, h=6 m.

H(m)	P _{us}	P _{ud}	Pu (kN/m)	y (m)	P
7	3830.40	18287.50	3830.40	0.000	0.00
				0.002	540.40
				0.004	1054.88
				0.006	1522.26
				0.008	1929.13
				0.010	2270.37
				0.015	2858.43
				0.020	3167.08
				0.025	3317.31
				0.030	3387.74
				0.040	3435.00
				0.050	3444.81
				0.100	3447.36
				0.250	3447.36
				0.500	3447.36

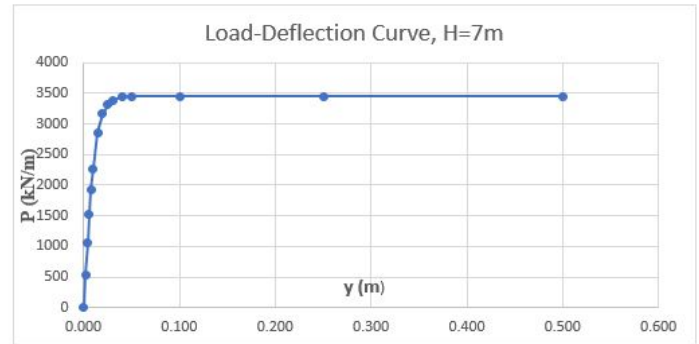


Figure 6.11. Typical multilinear plastic link property defined for p-y curve, h=7 m.

H(m)	P _{us}	P _{ud}	Pu (kN/m)	y (m)	P
8	4818.40	20900.00	4818.40	0.000	0.00
				0.002	618.49
				0.004	1212.32
				0.006	1760.61
				0.008	2248.89
				0.010	2669.90
				0.015	3434.94
				0.020	3872.08
				0.025	4103.63
				0.030	4221.37
				0.040	4308.88
				0.050	4329.96
				0.100	4336.55
				0.250	4336.56
				0.500	4336.56

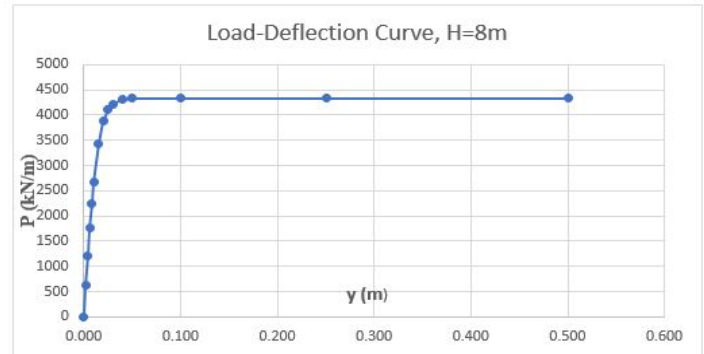


Figure 6.12. Typical multilinear plastic link property defined for p-y curve, h=8 m.

H(m)	P _{us}	P _{ud}	Pu (kN/m)	y (m)	P
9	5916.60	23512.50	5916.60	0.000	0.00
				0.002	696.56
				0.004	1369.69
				0.006	1998.99
				0.008	2569.38
				0.010	3072.04
				0.015	4025.49
				0.020	4609.80
				0.025	4942.12
				0.030	5123.15
				0.040	5270.04
				0.050	5310.15
				0.100	5324.92
				0.250	5324.94
				0.500	5324.94

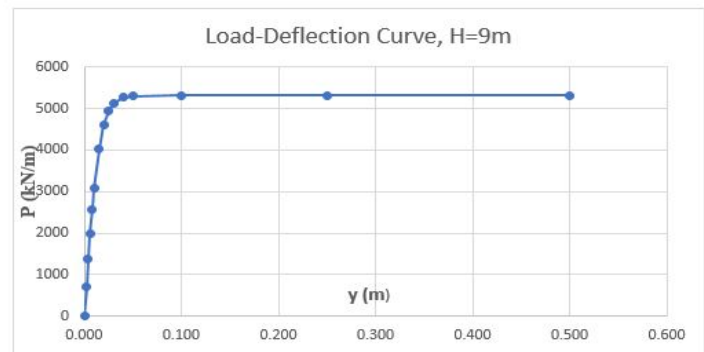


Figure 6.13. Typical multilinear plastic link property defined for p-y curve, h=9 m.

H(m)	P _{us}	P _{ud}	Pu (kN/m)	y (m)	P
10	7125.00	26125.00	7125.00	0.000	0.00
				0.002	774.62
				0.004	1526.96
				0.006	2237.22
				0.008	2890.04
				0.010	3475.45
				0.015	4625.59
				0.020	5372.70
				0.025	5824.10
				0.030	6084.98
				0.040	6313.44
				0.050	6382.91
				0.100	6412.43
				0.250	6412.50
				0.500	6412.50

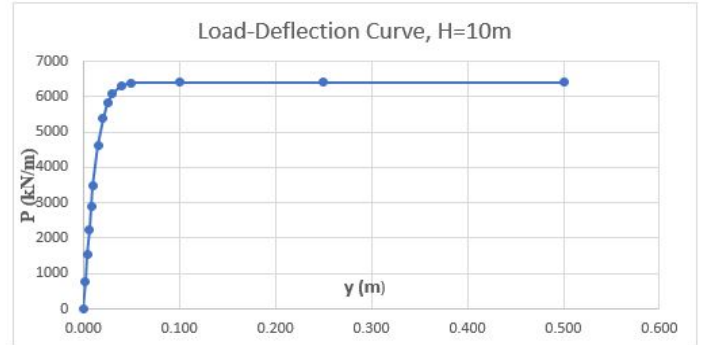


Figure 6.14. Typical multilinear plastic link property defined for p-y curve, h=10 m.

H(m)	P _{us}	P _{ud}	Pu (kN/m)	y (m)	P
11	8443.60	28737.50	8443.60	0.000	0.00
				0.002	852.66
				0.004	1684.11
				0.006	2475.22
				0.008	3210.54
				0.010	3879.30
				0.015	5232.00
				0.020	6154.72
				0.025	6741.80
				0.030	7098.96
				0.040	7433.42
				0.050	7545.10
				0.100	7599.05
				0.250	7599.24
				0.500	7599.24

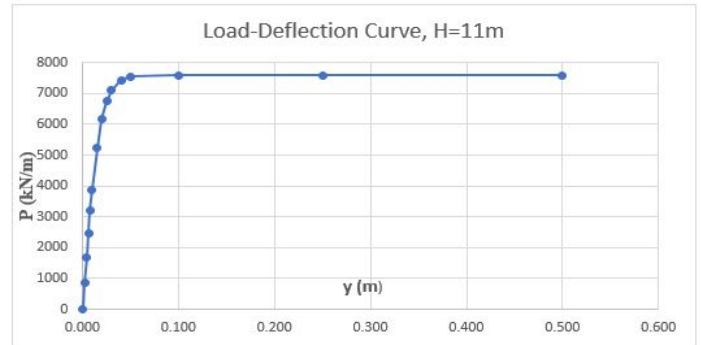


Figure 6.15. Typical multilinear plastic link property defined for p-y curve, h=11 m.

H(m)	P _{us}	P _{ud}	Pu (kN/m)	y (m)	P
12	9872.40	31350.00	9872.40	0.000	0.00
				0.002	930.68
				0.004	1841.16
				0.006	2712.95
				0.008	3530.71
				0.010	4283.11
				0.015	5842.42
				0.020	6950.99
				0.025	7688.49
				0.030	8157.70
				0.040	8623.97
				0.050	8793.00
				0.100	8884.68
				0.250	8885.16
				0.500	8885.16

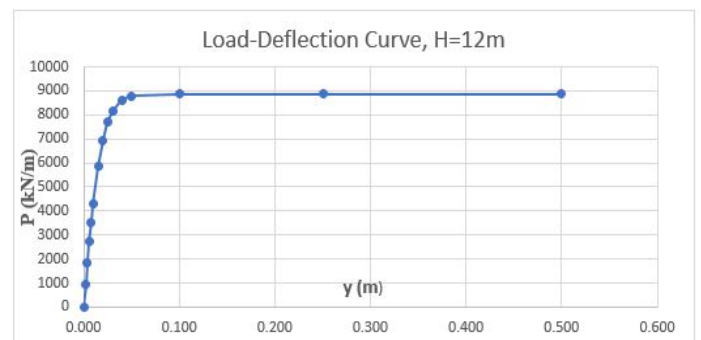


Figure 6.16. Typical multilinear plastic link property defined for p-y curve, h=12 m.

H(m)	P _{us}	P _{ud}	P _u (kN/m)	y (m)	P
13	11411.40	33962.50	11411.40	0.000	0.00
				0.002	1008.68
				0.004	1998.09
				0.006	2950.40
				0.008	3850.45
				0.010	4686.56
				0.015	6455.20
				0.020	7757.73
				0.025	8658.41
				0.030	9254.41
				0.040	9878.89
				0.050	10122.39
				0.100	10269.18
				0.250	10270.26
				0.500	10270.26

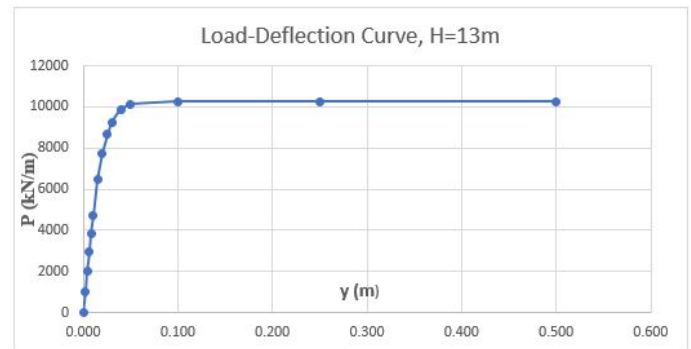


Figure 6.17. Typical multilinear plastic link property defined for p-y curve, h=13 m.

H(m)	P _{us}	P _{ud}	P _u (kN/m)	y (m)	P
14	13060.60	36575.00	13060.60	0.000	0.00
				0.002	1086.68
				0.004	2154.94
				0.006	3187.59
				0.008	4169.73
				0.010	5089.50
				0.015	7069.20
				0.020	8571.98
				0.025	9646.73
				0.030	10383.03
				0.040	11192.01
				0.050	11528.69
				0.100	11752.33
				0.250	11754.54
				0.500	11754.54

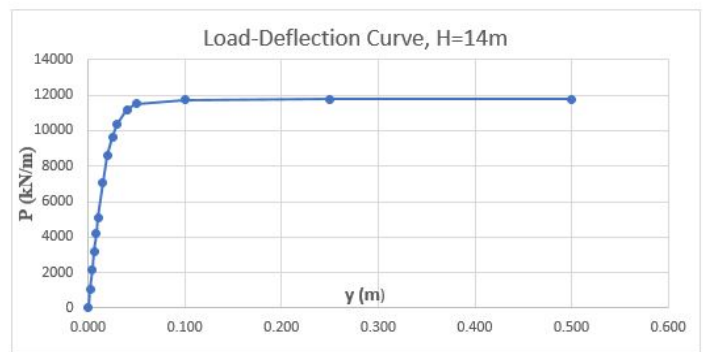


Figure 6.18. Typical multilinear plastic link property defined for p-y curve, h=14 m.

H(m)	P _{us}	P _{ud}	P _u (kN/m)	y (m)	P
15	14820.00	39187.50	14820.00	0.000	0.00
				0.002	1164.66
				0.004	2311.69
				0.006	3424.52
				0.008	4488.55
				0.010	5491.83
				0.015	7683.63
				0.020	9391.49
				0.025	10649.48
				0.030	11538.22
				0.040	12557.33
				0.050	13007.05
				0.100	13333.79
				0.250	13338.00
				0.500	13338.00

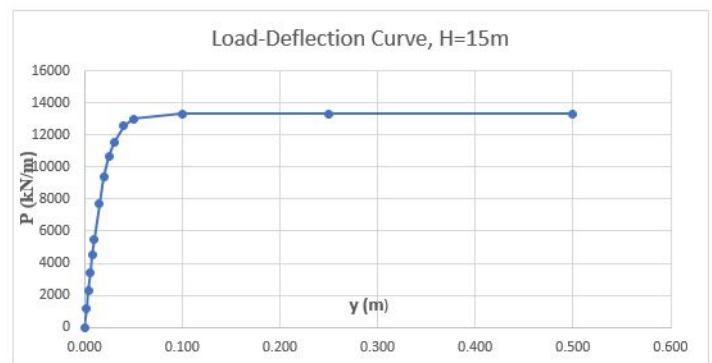


Figure 6.19. Typical multilinear plastic link property defined for p-y curve, h=15 m.

6.2. Mode Shapes and Periods for 15 m Cap Beam Width

6.2.1. Mode Shape and Period for No Soil Embedment Case

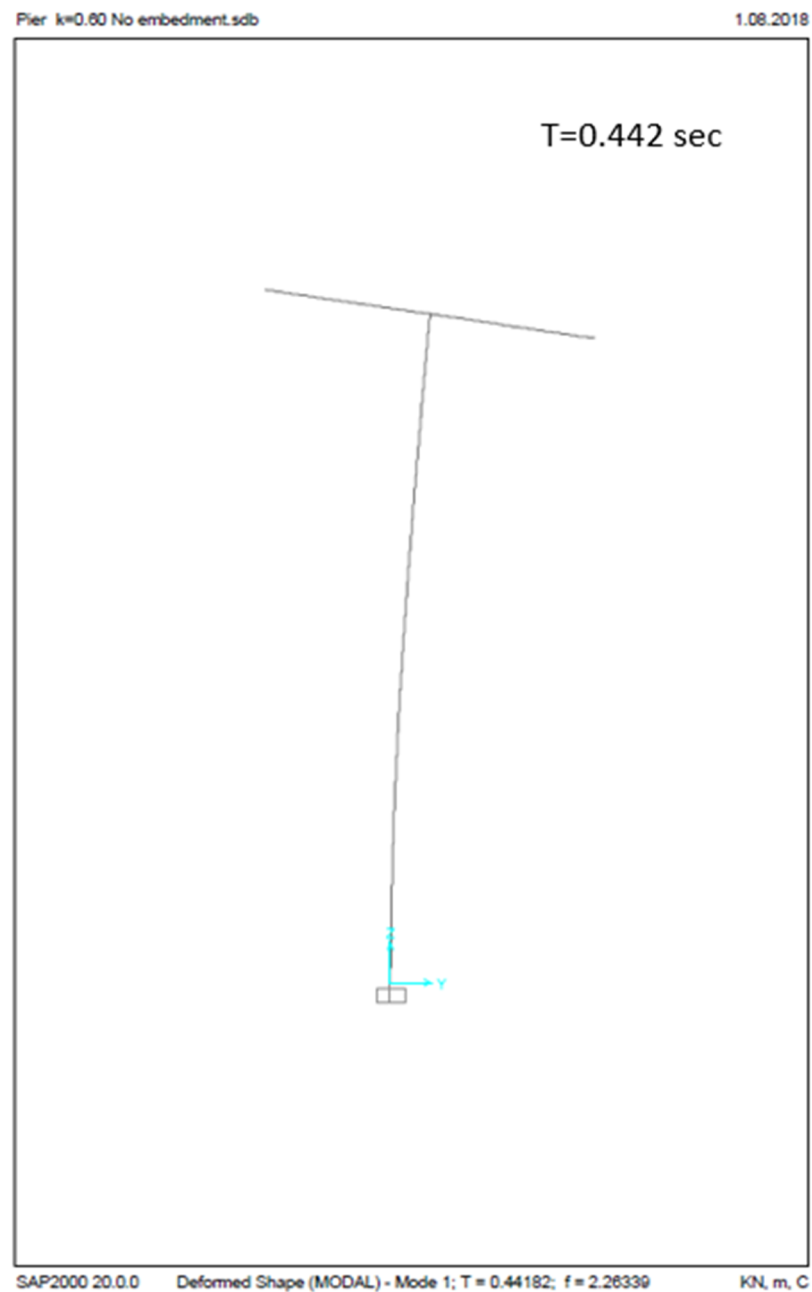


Figure 6.20. No soil embedment mode shape and period.

6.2.2. Mode Shape and Period for 5 m Soil Embedment Case

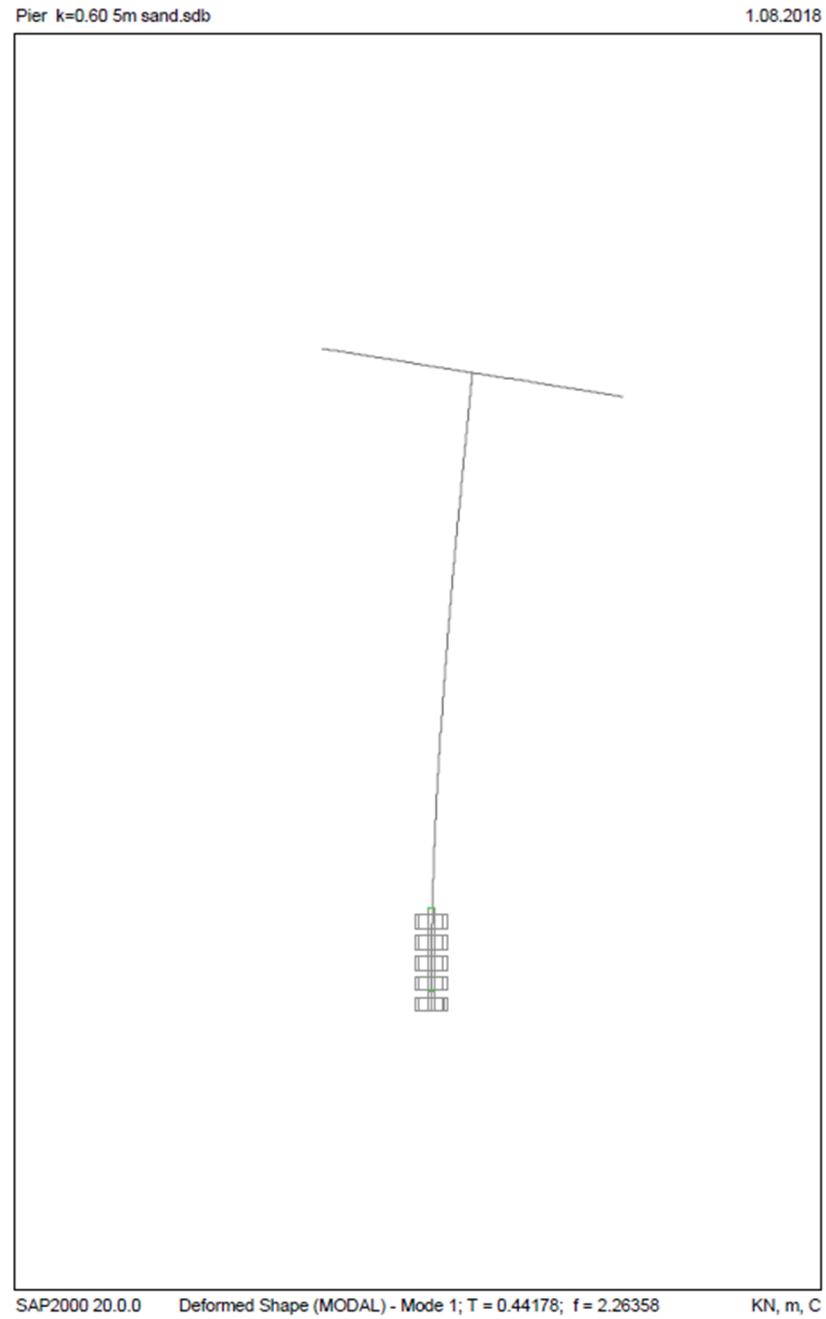


Figure 6.21. 5m soil embedment mode shape and period.

6.2.3. Mode shape and period for 10 m Soil Embedment case

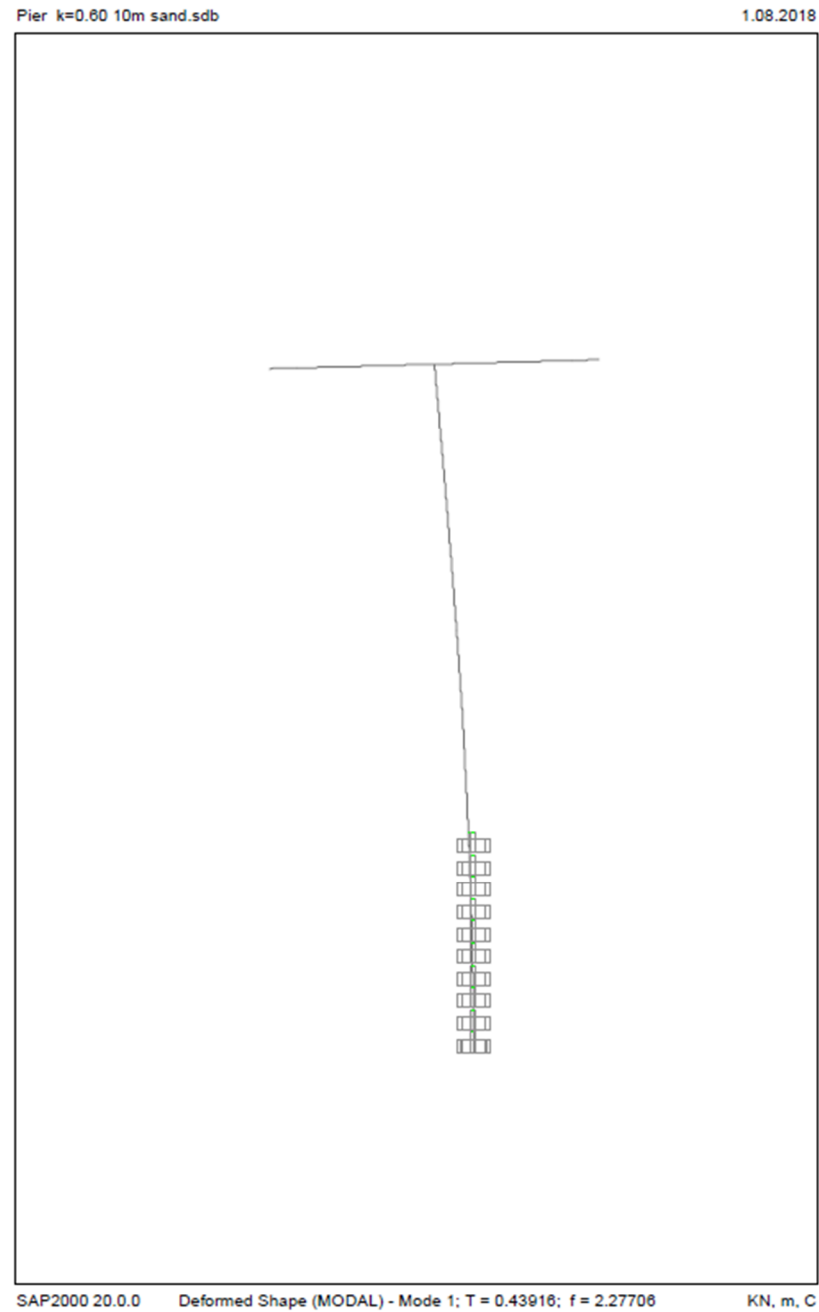


Figure 6.22. 10 m soil embedment mode shape and period.

6.2.4. Mode Shape and Period for 15 m Soil Embedment

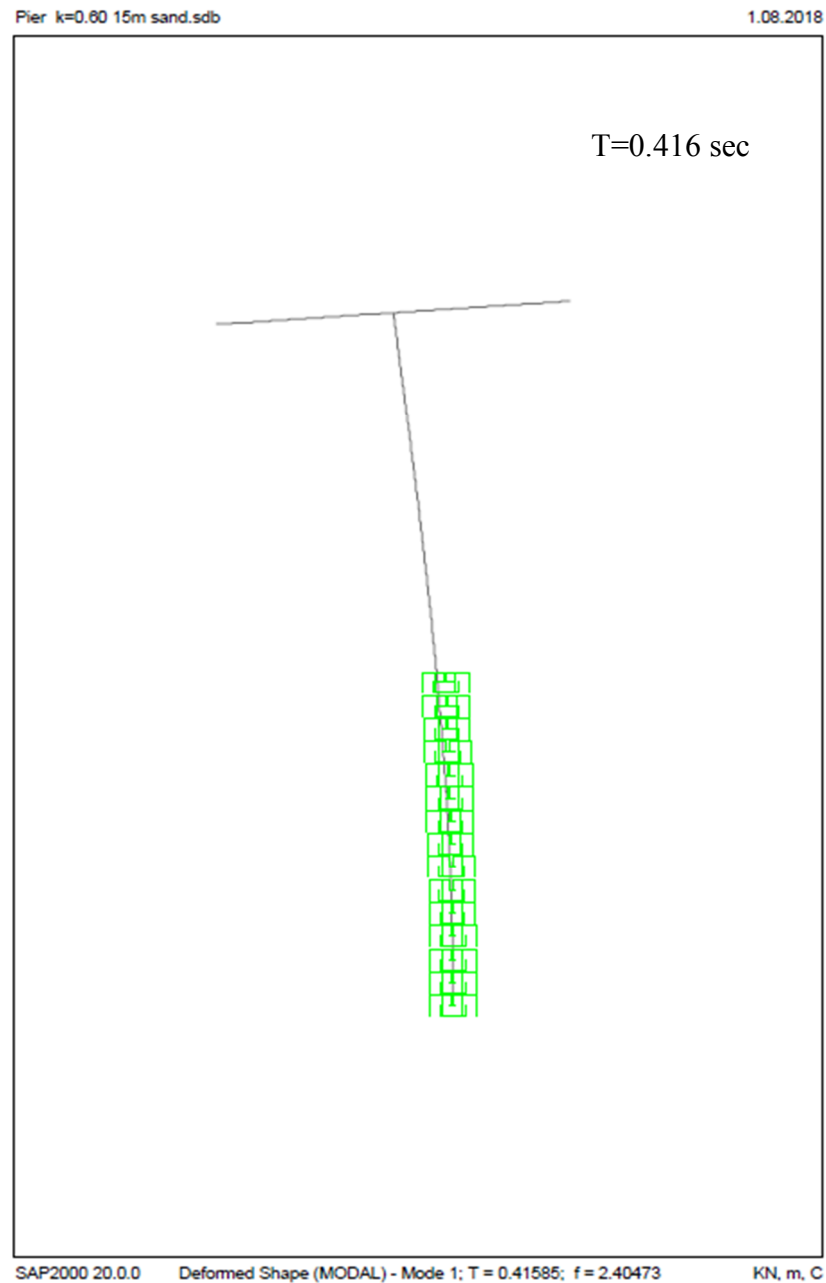


Figure 6.23. 15 m soil embedment mode shape and period.

6.3. Spectral Accelerations for Modal Periods in case of Different Soil embedment Depths

Elastic acceleration response spectra of horizontal and vertical ground motions are obtained from Kobe (1995-station: Port Island), Kocaeli (1999-station: Sakarya), Kocaeli (1999-station: Yarimca) records using Seismosignal which can be seen in Figure 6.24., Figure 6.25. and Figure 6.26. respectively. Spectral acceleration values for the modal period of 0.44 sec in case of no soil embedment and modal period of 0.42 sec in case of 15 m of soil embedment depth are shown on the response spectra.

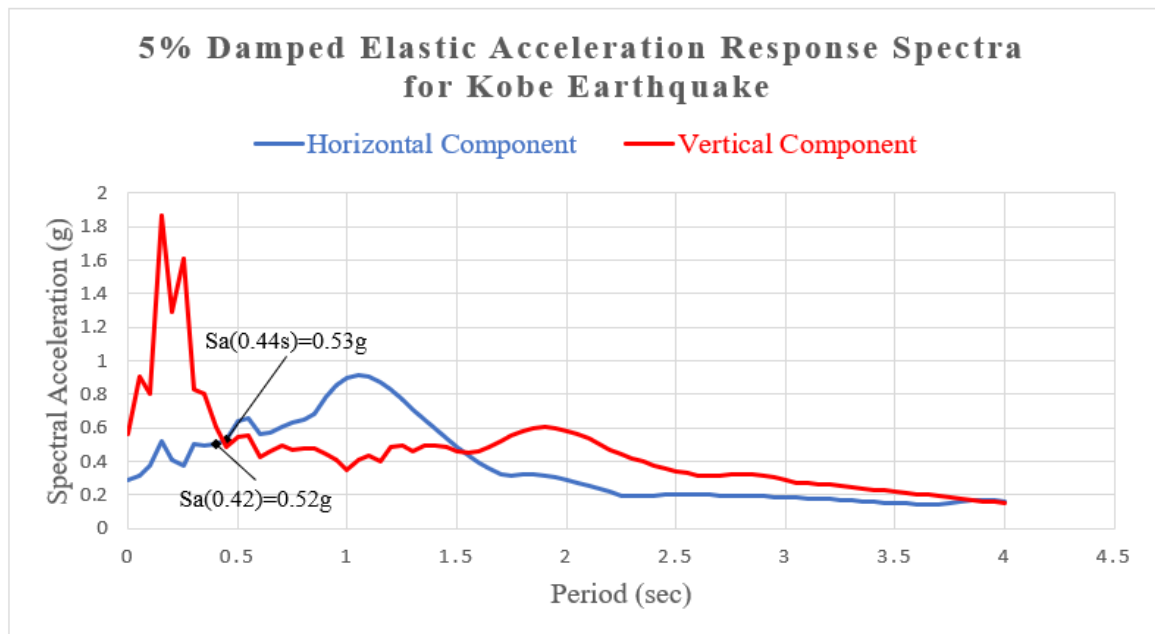


Figure 6.24. 5% damped elastic acceleration response spectra for Kobe earthquake vertical and horizontal component, the dots represent the spectral acceleration values for modal periods in case of no soil embedment and 15 m soil embedment.

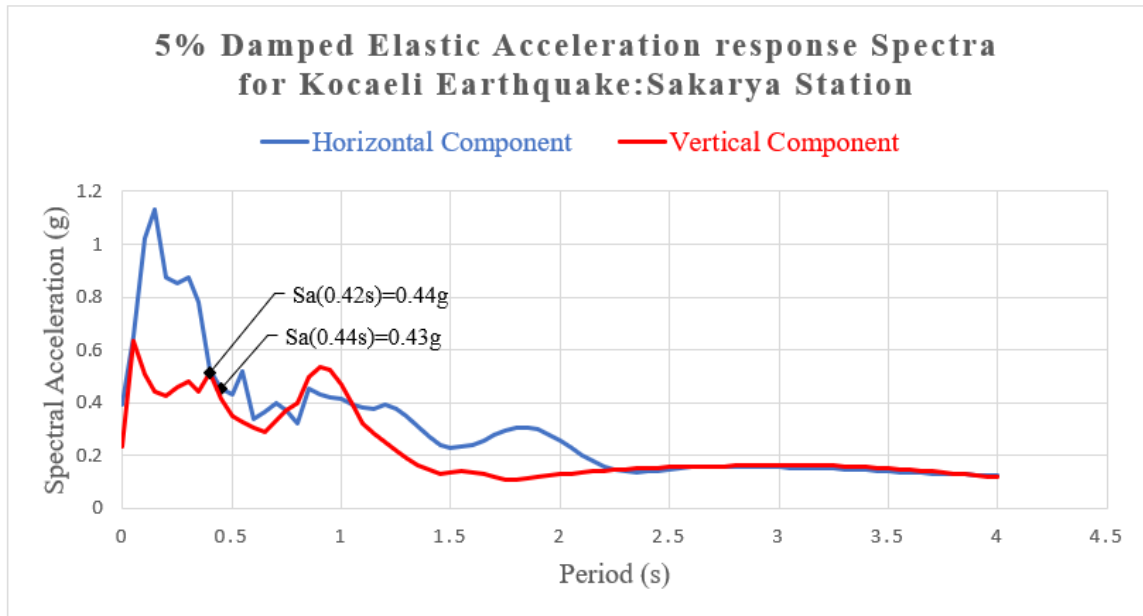


Figure 6.25. 5% damped elastic acceleration response spectra for Sakarya station Kocaeli earthquake vertical and horizontal component, the dots represent the spectral acceleration values for modal periods in case of no soil embedment and 15 m soil embedment.

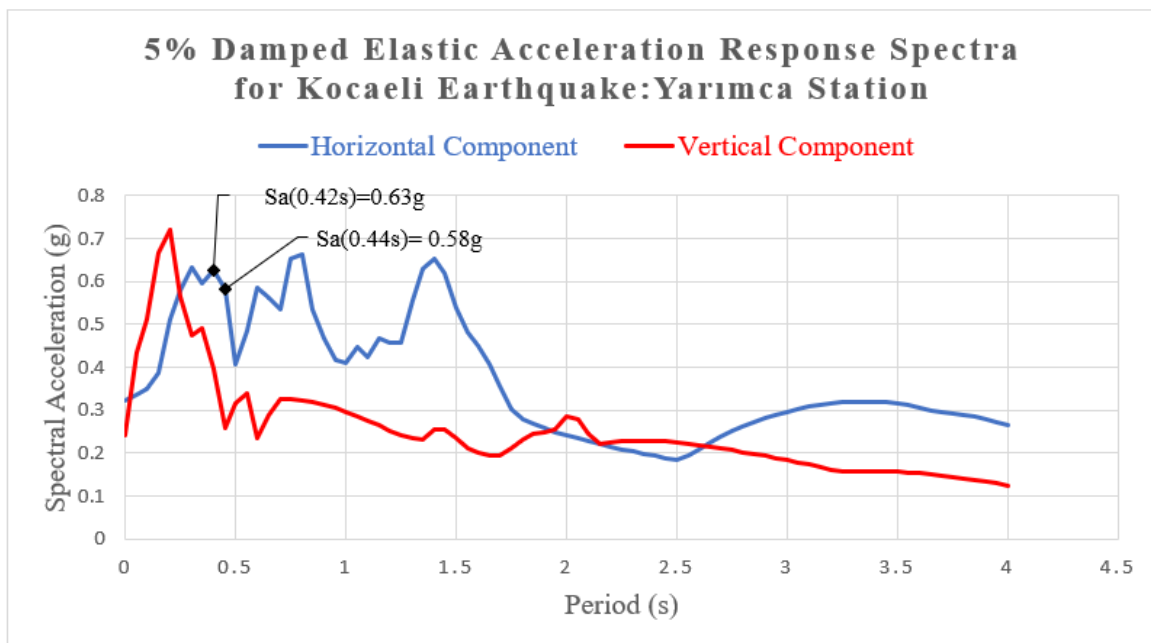


Figure 6.26. 5% damped elastic acceleration response spectra for Yarimca station Kocaeli earthquake vertical and horizontal component, the dots represent the spectral acceleration values for modal periods in case of no soil embedment and 15 m soil embedment.

6.4. Shear force Diagrams for Different Soil Embedment Depths

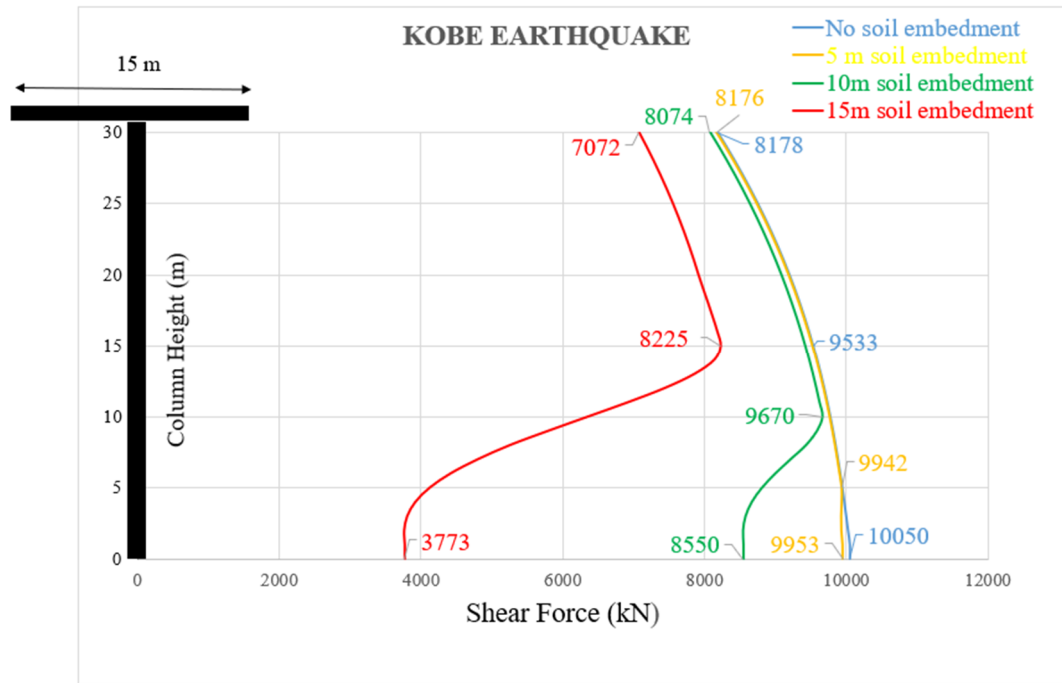


Figure 6.27. Shear force diagrams: Kobe earthquake.

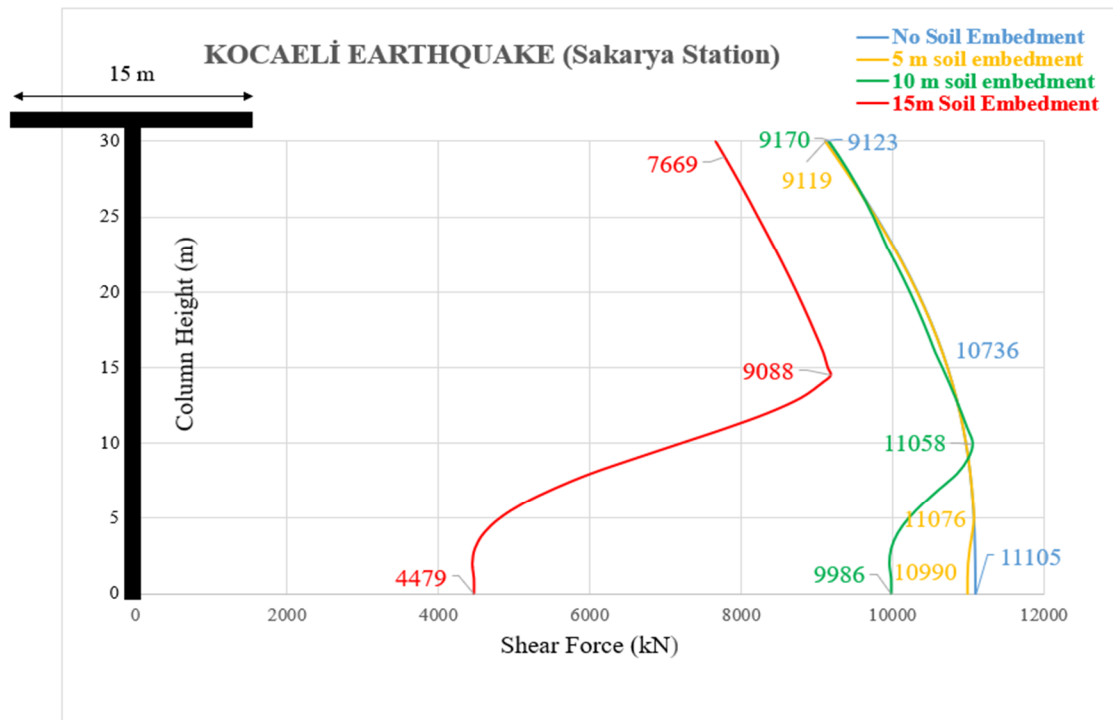


Figure 6.28. Shear force diagrams: Sakarya station- Kocaeli earthquake.

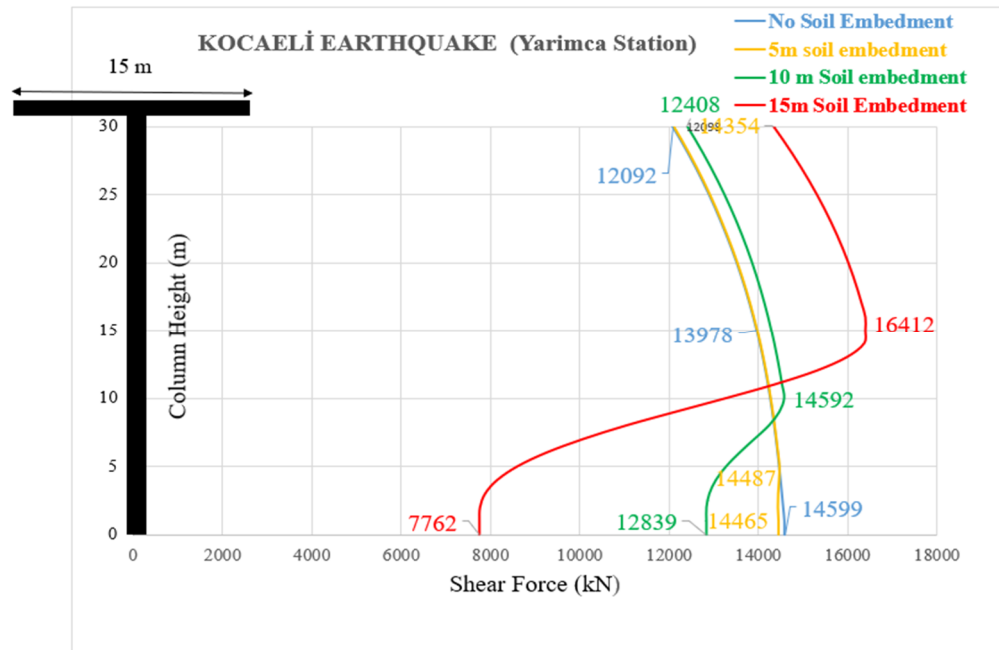


Figure 6.29. Shear force diagrams: Yarimca station Kocaeli earthquake.

6.5. Moment Diagrams for Different Soil Embedment Depths

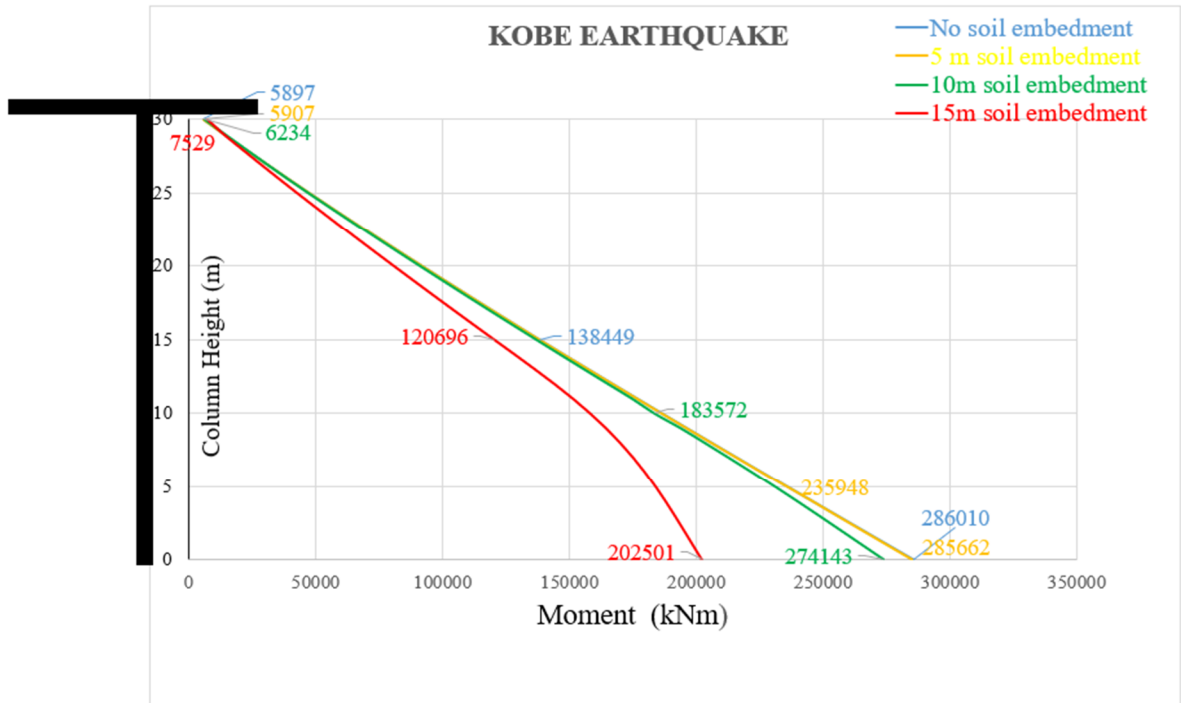


Figure 6.30. Moment diagrams: Kobe earthquake.

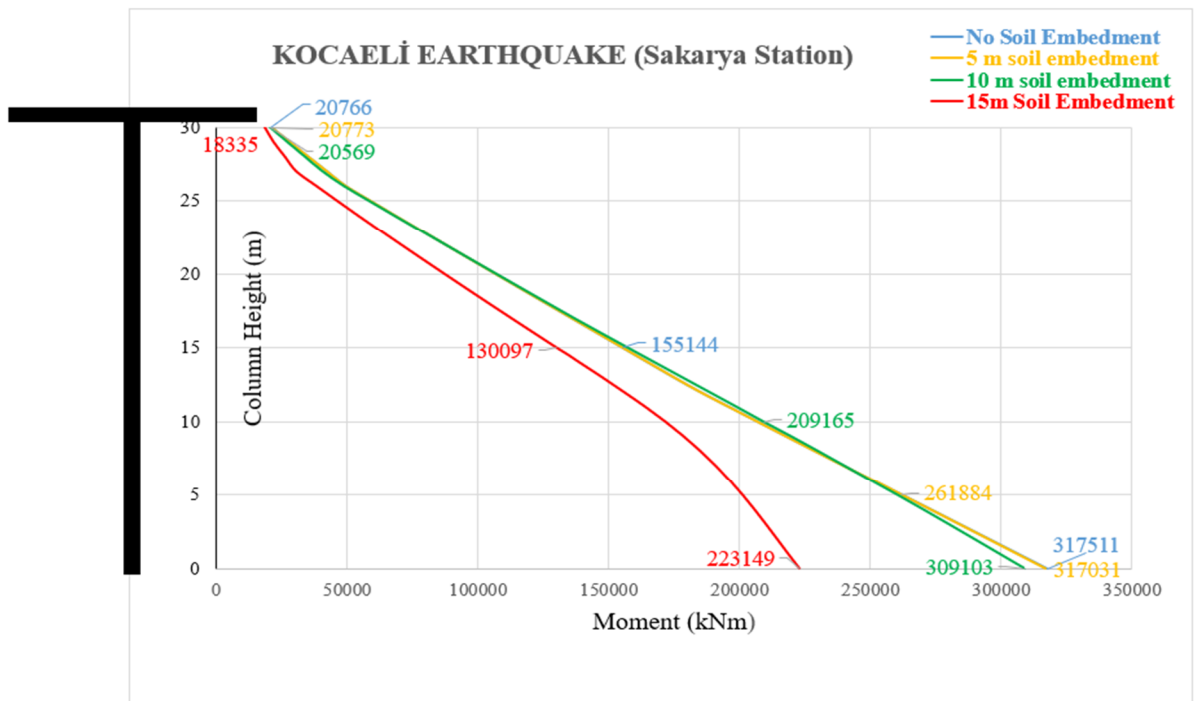


Figure 6.31. Moment diagrams: Sakarya Station Kocaeli earthquake.

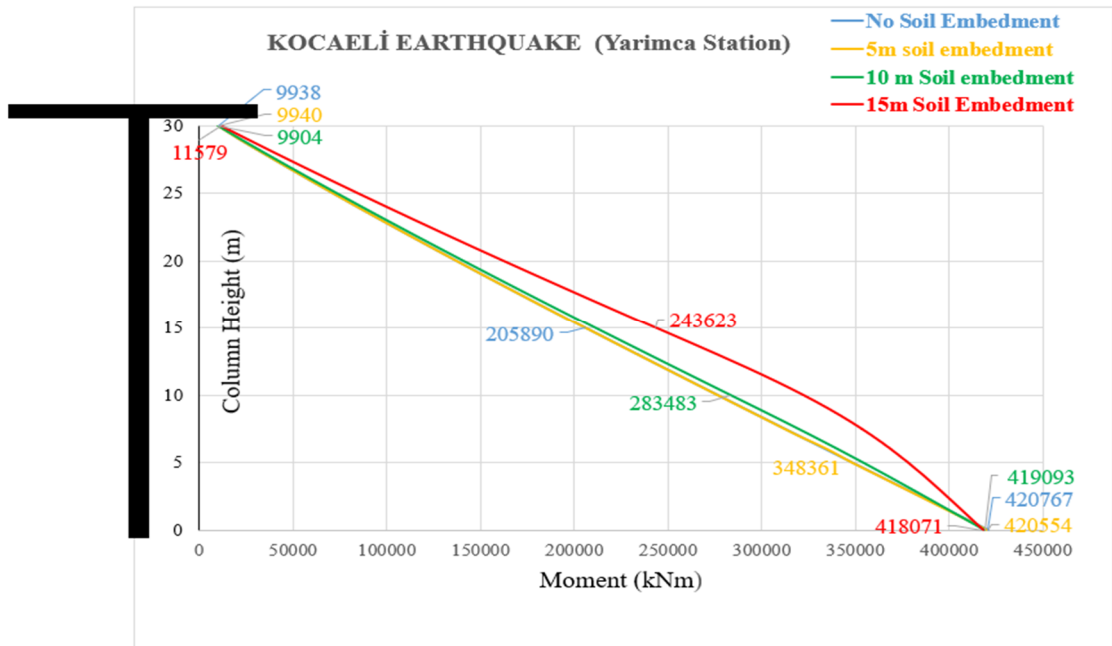


Figure 6.32. Moment diagrams: Yarimca station Kocaeli earthquake.

7. CONCLUSIONS

The main purpose of this study is to investigate the effects of vertical component of the earthquake ground motions on the behaviour of a single-column, RC bridge pier. The analyses are conducted on an RC bridge pier of 30 m height with a 15 m-wide cap beam. Time history analyses are performed for two mathematical models under the selected three earthquake recordings; i) only horizontal component of the earthquake records and ii) both horizontal and vertical components. The dynamic response of the pier is examined for different cases where varying cap beam to column stiffness ratios are adopted. The flexural stiffness ratios of cap beam to column used in the analysis are 0.3, 0.6 and 0.9 respectively. Cap beam cross section is modified accordingly to obtain these flexural stiffness ratios for each analysis case. Column moment and shear forces are compared for eighteen analysis cases, i.e. only horizontal components and, horizontal and vertical components acting together are considered for three earthquake recordings and for three different flexural stiffness ratios of cap beam to column.

When the pier is subjected to horizontal and vertical components of Kocaeli earthquake-Sakarya station record simultaneously, column base shear forces increase 4.5 %, 5.6 %, 6.6 % for the flexural stiffness ratios of cap beam to column 0.30 – 0.60 – 0.90 respectively with respect to horizontal-only ground motion. When the pier is subjected to the horizontal and vertical ground motion of Kobe earthquake simultaneously, the shear force at the mid height of column increases 3.9 %, 4.1 %, 5.2 % for the flexural stiffness ratios of cap beam to column 0.30 – 0.60 – 0.90 respectively with respect to horizontal-only ground motion. The maximum increase in the column base shear forces developed under Kocaeli earthquake Yarimca record. When the pier is subjected to the horizontal and vertical ground motion simultaneously, column base shear forces increase 2.2 % and 2.8 %, for the flexural stiffness ratios of cap beam to column 0.30 – 0.6 with respect to horizontal-only ground motion. In the case of the flexural stiffness ratio of cap beam to column 0.90, column base shear forces increase 8.9 % when the column is subjected to both horizontal and vertical ground motion with respect to horizontal-only ground motion.

It is seen that P- Δ effect causes a significant increase in the shear force for the flexural stiffness ratio of 0.90 in the analyses cases in which the horizontal and vertical components of the earthquake ground motions are acting simultaneously on the bridge pier. On the other hand, any increase in the column bending moments is not observed for that case once compared with horizontal-only ground motion analyses cases.

In the second part of the study, the seismic response of the same pier is assessed when it is buried under 5 m, 10 m and 15 m of earth fill. The change in moment and shear force are evaluated when the certain part of the pier to remain under the soil filling material. In case of no soil embedment maximum moment and shear forces due to seismic forces develops at the bottom of the column. In case of soil embedment column moment and shear force values increase at upper level of vertical embedment length. Shear force and bending moment increases 17.5 % and 18 % respectively at the upper level of 15 m of soil embedment with respect to no soil embedment under Kocaeli earthquake-Yarimca station record. The most critical situation for the column forces occurs under Kocaeli Earthquake-Yarimca station. Soil embedment causes relatively lesser moment and shear forces along the column under Kobe earthquake and Kocaeli earthquake-Sakarya station.

The results show that the seismic response of the RC bridge pier in case of different soil embedment conditions is closely related to the strong motion characteristics of the selected earthquake records. For better identification of its behaviour more analyses under different earthquake recordings are needed. In addition, to investigate the effect of soil embedment at bridge piers, modelling the overall bridge and performing the seismic analysis on the bridge model may reveal significant increase of shear forces at the buried pier. Shortening the free column height due to the soil embedment may change the shear force distribution between tall and short columns. Due to the increased stiffness of the buried pier with regard to longer pier next to it, the buried pier may subject to greater shear force demand. In largescale construction projects, after the construction phase is completed, as built projects are offered. In the case of submitted as-built projects, if the columns are buried in the soil embedment, the analyses should be repeated to check whether the columns should be reinforced.

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