A SIMPLE ASSESSMENT OF LATERAL PIER RESPONSE OF STANDARD HIGHWAY BRIDGES ON PILE FOUNDATIONS

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I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

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ABSTRACT

A SIMPLE ASSESSMENT OF LATERAL PIER RESPONSE OF STANDARD HIGHWAY BRIDGES ON PILE FOUNDATIONS

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Group of piles are widely used deep foundation systems to resist lateral and vertical loads. Seismic and static performance of pile groups mostly depend on soil type, pile spacing and pier rigidity.

Not many pile lateral load tests have been performed due to high costs. Advanced and complex analytical methods were developed over the years to assess nonlinear lateral pile response. This research is conducted aiming at developing a practical analysis method to verify the lateral performance of pile groups and its effect on overall response of bridge utilizing the available pile lateral load test data. Empirical constants derived from evaluation of lateral load tests are used in a simple formulation to define the nonlinear behavior of the pile-soil system. An analysis guideline is established to model the nonlinear soil-bridge interaction by the help of a general purpose structural analysis program comprising recommendations for various cases. Results of the proposed method is compared to the results of industry accepted advanced methods using response spectrum and nonlinear time history analyses to assess the suitability of this new application. According to the analysis results, proposed simple method can be used as an effective analysis tool for the determination of response of the superstructure.

Keywords: Lateral response, soil-structure interaction, pile foundation, pile load test, soil nonlinearity

ÖZ

KAZIKLI STANDART KARAYOLU KÖPRÜLERİNDE YATAY KOLON DAVRANIŞININ BASİT YÖNTEMLE TESPİTİ

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Düşey ve yatay yükler için genelde kazık grupları derin temel sistemi olarak kullanılmaktadır. Kazık gruplarının sismik ve statik yükler altındaki performansı genelde zemin tipine, kazıklar arası mesafeye ve yapı taşıyıcı sisteminin rijitliğine bağlıdır.

Yüksek maliyetinden ötürü çok fazla yatay kazık yükleme deneyi yapılamamaktadır. Yıllar boyunca doğrusal olmayan yatay kazık davranışını açıklamak amacıyla gelişmiş ve karmaşık analitik yöntemler geliştirilmiştir. Burada, kazık gruplarının yatay peformansını ve köprü üzerindeki etkisini belirleyebilecek basit bir yöntem geliştirmek amacıyla bir çalışma yürütülmüştür. Mevcut yatay kazık yükleme deney sonuçları kullanılarak bulunan ampirik değerler doğrusal olmayan yapı-zemin etkileşimini belirlemek amacıyla basit bir formülasyonda kullanılmıştır. Zemin-köprü etkileşimini genel amaçlı bir yapısal analiz programında değişik durumlar için modellemek amacıyla bir analiz yöntemi oluşturulmuştur. Basit metodun kullanılabilirliği, literatürdeki diğer yöntemler ile örnek bir köprü modeli üzerinde spektral analiz ve zaman tanım alanında doğrusal olmayan hesap yöntemi kullanılarak yapılan kıyaslama sonucu belirlenmiştir. Analiz sonuçları burada oluşturulan basit metodun, üstyapı elemanlarının depreme karşı olan tepkisinin bulunmasında etkili bir şekilde kullanılabileceğini göstermiştir.

Anahtar Sözcükler: Yatay davranış, yapı-zemin etkileşimi, kazıklı temel, kazık yükleme deneyi, doğrusal olmayan zemin davranışı

To my family and my love Yeşim

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CHAPTER 1

INTRODUCTION

1.1 Background

Depending on circumstances, piles can be economical way of building a foundation. Since the drilling and driving machines are becoming so powerful, construction of piles is getting more popular day by day. Pile foundations transmit the superstructure loads to a stronger layer of soil below the ground level for the case of end bearing piles and distribute the loads from superstructure to the surrounding soils throughout the length of piles for the case of floating (friction) piles. Piles are particularly effective when the foundation soils are very soft, liquefiable or highly compressible. They are also preferred to support the structure against lateral loads induced by earthquake, wind and thermal forces.

However, pile foundations are more expensive compared to shallow foundations in general. As pile foundations cause considerable increases in project costs, many project owners and consultants throughout the world require full scale testing in order to check the design.

Lateral behavior of piles in different soil conditions are still not known very well and some empirical methods, most of which result in overdesign, are used to assess the lateral capacity. The best alternative for the most effective design is full scale lateral load testing at the field. In many countries both vertical and lateral load testing on piles are done before the final design of piles. However, in Turkey vertical tests are rarely performed and almost no lateral tests are conducted since testing is expensive and time consuming.

Some approximate methods were developed to assess the static and dynamic pile response. However, most of these methods disagree with each other mainly due to the uncertainties involved in the geotechnical data. Pile behavior can be modeled by complicated or simple methods. In the presence of too many uncertainties, it will be practical to use simplified methods rather than complicated methods. To overcome the uncertainties in the approximate methods, more testing is required. When lateral loads are considered, increasing number of tests would, no doubt, describe the effect of soil on piles better.

There exist numerous studies regarding the assessment of pile behavior under lateral loads, which can be classified in three broad categories. First one is “full-scale testing” which is the most effective for the assessment of pile behavior, but also the most expensive. Second approach is “model tests” performed in laboratory conditions. This type of study can be carried out using small scale piles either by application of the loads directly or by utilizing a centrifuge device. Last type of approach is “analytical solutions” which is time efficient and economical. In fact, a number of load tests are also required for this type of approach in order to check whether the nonlinear behavior of soil and its effect on pile is modeled properly.

1.2 Aim of The Study

Since testing of piles under lateral loads is rather expensive and the available methods in literature require many laboratory tests for the determination of soil parameters, a simple method that will be used for design process might be developed. Hence, the main objective of this study is development of a simplified method for determination of nonlinear load deflection behavior of pile foundations. This simple approach is intended to rely on only the Standard Penetration Test blow counts (SPT-N) and no laboratory testing will be required. In order to recommend a formulation, available lateral load test results from literature will be used for back calculation purposes and certain empirical constants will be introduced based on the results attained.

In the light of this information, being the most dependable approach, a number of full scale tests conducted on single pile or group of piles will be discussed in the following chapter. In addition to that, the analytical methods cited in some specifications will also be examined.

CHAPTER 2

LITERATURE SURVEY

2.1 Earlier Studies

The evolution of the lateral load-deflection analysis of piles goes back to the late 1940ies and 1950ies. It was the era of development of the energy companies which built offshore structures to benefit from the submarine petroleum reserves. As would be expected, these structures receive high horizontal loads particularly during severe storms. In order to design offshore structures safely against lateral loads, a diversity of models and analysis methods have been proposed by researchers.

During late 1940s, the standard beam on elastic foundation equations, suggested by Hetenyi, had been used for modelling of a pile subjected to lateral loading. Later, Terzaghi (1955) suggested the “subgrade modulus” concept, to design piles both for deflection and bending moment. On the other hand, Terzaghi also stated that utilization of this approach would be questionable in case of the loading exceeding half of the soil bearing capacity, but he gave no recommendation for calculation of soil bearing capacity for a laterally loaded pile. Subsequently, many recommendations have been made for subgrade modulus values for different soil types. Using values from tables providing subgrade reaction modulus is the easiest way of modelling pile response, since lateral soil resistance is described simply by springs. Bowles recommended a range of subgrade reaction modulus values for some specific soils. As it can be seen in Table 2.1, the suggested values display rather high ranges and accordingly, the analysis may result in widely different results for the same type of soil.

Table 2.1. Range of modulus of subgrade reaction k_s (Bowles)

Soil Type	**k_s (kN/m^3)**
Loose Sand	4800 – 16000
Medium dense sand	9600 – 80000
Dense Sand	64000 – 128000
Clayey medium dense sand	32000 – 80000
Silty medium dense sand	24000 – 48000
Clayey soil:	
$q_a \leq 200$ kPa	12000 – 24000
$200 < q_a \leq 800$ kPa	24000 – 48000
$q_a > 800$ kPa	> 48000

Apart from these studies, elastic modelling has been investigated by Poulos & Davis (1980) for different cases of single piles and pile groups. But for large loads, their model was not effective due to the fact that the soil behaves nonlinearly when the lateral force on the pile is increased. However, for the calculation of some simple cases, they presented empirical formulas regarding calculation of the subgrade reaction modulus for clays and sands based on soil properties. In 1993, American Petroleum Institute’s correlation between angle of friction of sands and modulus of subgrade reaction, as illustrated in Figure 2.1, was introduced and became a useful tool for analysis and design. Alternatively, Prakash and Kumar suggested that the horizontal subgrade reaction was an exponential function of the strain. Later, summarizing the findings of lateral load analysis of single piles and drilled shafts, Duncan (1994) stated that the relationship between load, deflection and moment for a laterally loaded pile is always nonlinear irrespective of the load level.

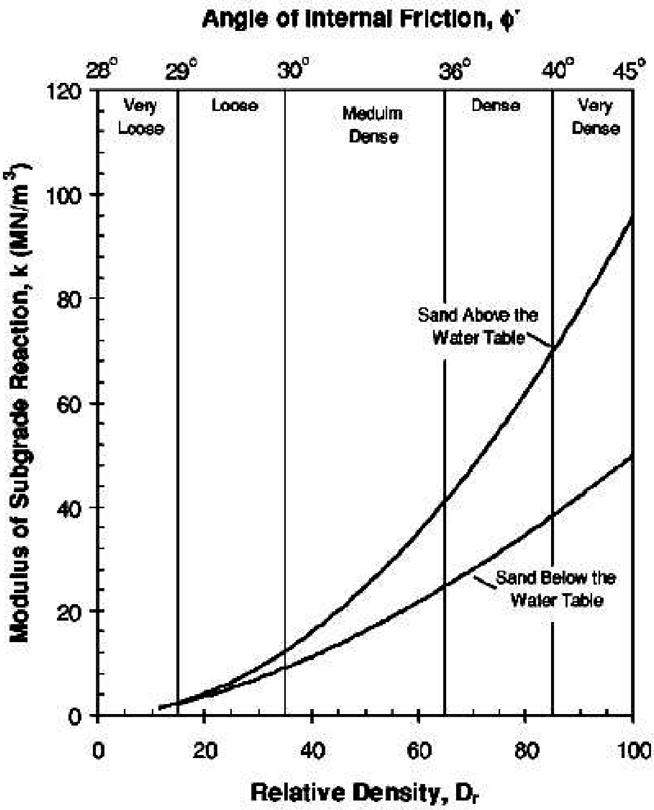


Figure 2.1. Recommendation for modulus of subgrade reaction for sands by API (1993)

Due to the inefficiency of elastic models, modelling of soil with inelastic behavior became popular. Broms (1964) suggested that the deflection of pile for any working load could be found by the use of a model with rigid pile and nonlinearly behaving soil and proposed equations for the construction of p-y curves for piles in sands. With the use of high speed computers, this method still provides a quick solution for initial design of pile foundations. As the computer technology was developing day by day, researchers began nonlinear modelling of the pile – soil system. In these models, both pile and soil are defined as nonlinearly behaving members.

Later, American Petroleum Institute (API) (1993) presented the recommendations for nonlinear models regarding construction of p-y curves to be used for the analysis of offshore petroleum platforms. These methods, which are summarized in Appendices A.1.3 and A.2.2, also used for the design of foundations of onshore structures.

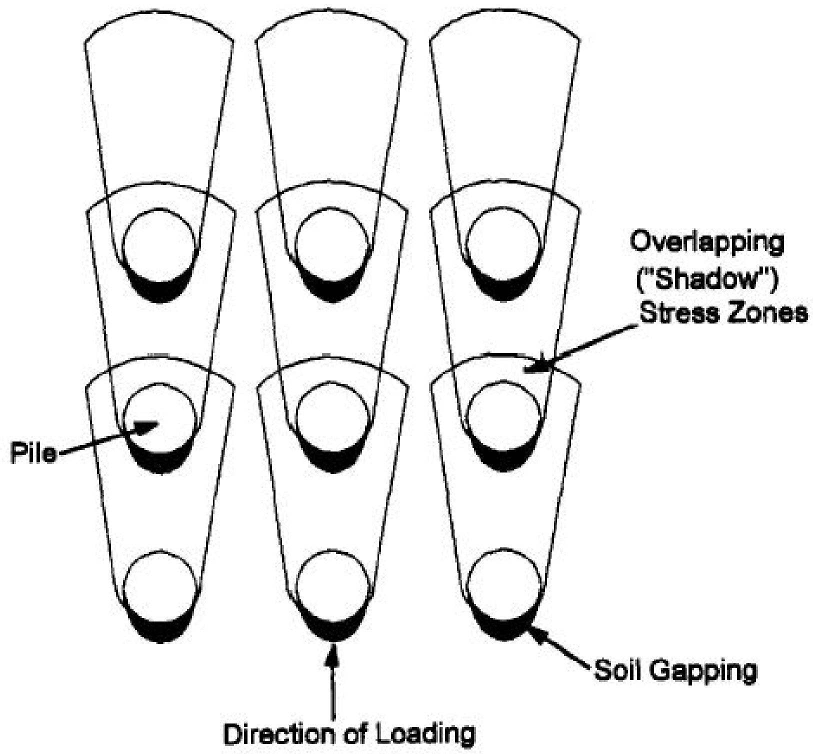


Figure 2.2. Schematic view of overlapping failure zones (Shadowing Effect) (Rollins, 1998)

Still, many researchers are conducting both full – scale and laboratory tests in order to relate the behavior of a pile in the group to different parameters such as loading conditions, group configuration, pile cap or pile properties. Pile group effect was studied by Ooi and Duncan (1994) through full – scale testing both on single steel pile and group of nine closely spaced steel piles. Spacing of the piles were three times the pile width, which is the most commonly used spacing in practice.

The deflection of the pile group was reported to be twice the deflection of the single pile when the same load per pile is considered and 20% higher bending moments were found for the piles in the group. It was also stated that the pile head condition was an important factor such that for the same lateral load, free head single pile deflected four times as that of the fixed head single pile. Also, the amplification factors for group was said to be a good approximation no matter what method is used for the behavior of single pile.

Another study on pile groups was conducted by Huang (2001). The research was based on testing of 13 cast–in–place bored piles and 13 precast concrete driven piles in a soil profile consisting of silty sand. The piles were three diameters far from each other. This study showed that the single pile behavior could be applied to the pile groups by using reduction factors called “p–multipliers”. Another important finding was the effect of construction method on p–multipliers as shown in Table 2.2

Table 2.2. P – Multipliers for bored and driven pile groups (Huang, 2001)

	Bored Group Piles	**Driven Group Piles**
Leading Row	0.932	0.893
Middle Row	0.704	0.614
Trailing Row	0.740	0.660

Concerning the group behavior, several full-scale tests were conducted by Rollins from 1990ies to 2006. In one case, Rollins worked on a 15 pile group in a silty and clayey soil. The load testing had been done on 324 mm diameter steel pipe piles arranged in a group of 3x5 with center–to–center spacing of 3.92 diameters. Although the piles were arranged as to form a group, each pile was attached to a frame with a pin connection. In determination of the effect of cyclic loading on the group, it was stated that the peak load on the pile was reduced by about %20 in 15 cycles. Major part of this reduction took place in the first few cycles and after the 10^{th} cycle there was no reduction. Another important result of these tests was related to the dynamic resistance of the pile group system. According to the test results, the dynamic resistance was 30% to 60% higher than the peak static resistance. Similarly, the area under the load – deflection curve for cyclic loading was larger than that of static loading. Regarding the group efficiency, concluding remarks of the paper was that the reduction factor was dependent on the row location of the pile in the group rather than the location within the same row. For the first row, the lateral resistance was larger and decreasing as the row position in the group increased up to approximately third row after which the amount of reduction became constant.

In another study, Rollins (2001) worked on the lateral response of 0.324 m outer diameter open ended driven steel pipe piles in sand. The pile group consisted of 9 piles arranged as 3x3 with center–to–center spacing of 3.3 pile diameters in both directions as shown in Figure 2.3. In this study, both single pile and the group were loaded and the results are shown graphically in Figure 2.4. As it can be observed, any row of the pile group deflects more than single pile under lateral loading. One interesting result of the test was that, as the row number increases there occurs greater reduction as would be expected, but the last row carries slightly more load than the previous (Figure 2.4). Rollins also investigated whether the piles in the same row resisted uniform lateral loads or not. Test results showed that the piles in the same row received different amounts of load. For a row of 3 piles, the piles on the sides carried 20 to 40 % higher loads than the pile in the middle, as shown in Figure 2.5. Moreover, it was stated that based on the previous testing by other researchers, p–multipliers increase with the increasing pile spacing. A value of 1.0 for a p–multiplier means that piles are widely spaced and there is no group effect present on the pile. This case generally occurs for spacing of 5 diameters for the first row, 6 diameters for second and third rows and 8 diameters for the fourth and higher row piles.

The concept of p–multipliers was reported by Ooi (2004) including a comment based on a comparison of the elastic solutions with nonlinear procedures.

The study consisted of the comparisons of full–scale pile load tests with each other. It was concluded that the reduction coefficients (p–multipliers) were dependent on position of row, pile spacing, soil type and installation method. Also, regarding elastic solutions, the piles at the corners received the maximum loads, whereas in reality front row is loaded more heavily than the following rows.

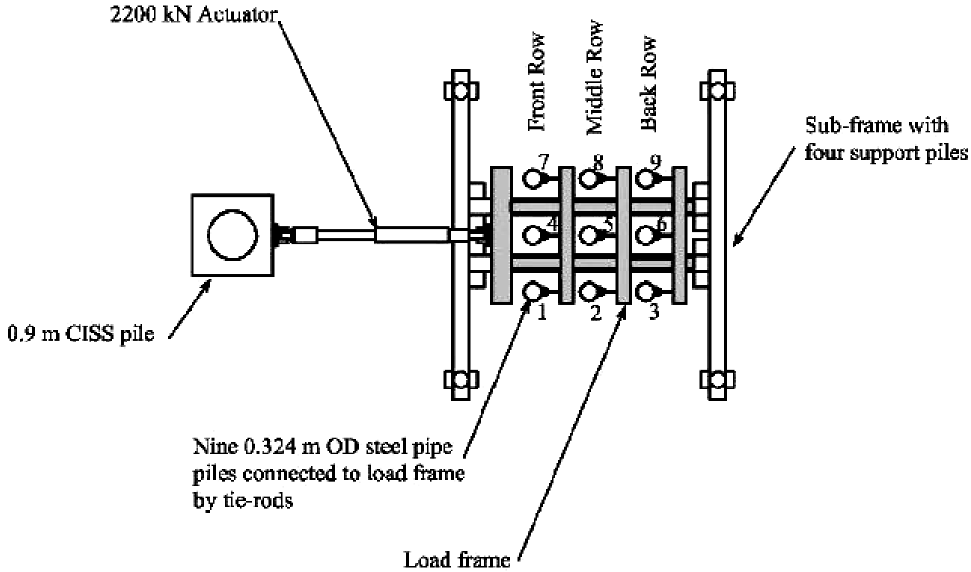


Figure 2.3. Lateral load test setup in the study of Rollins (2001)

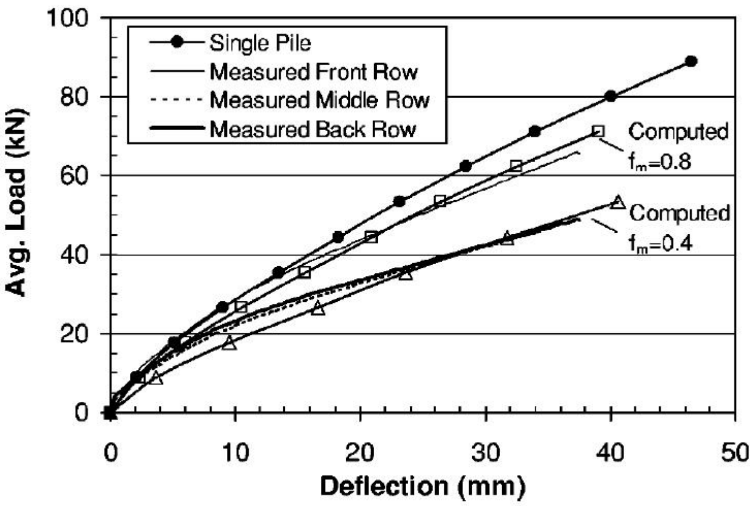


Figure 2.4. Average load – deflection curves for single pile and each row in the group (Rollins, 2001)

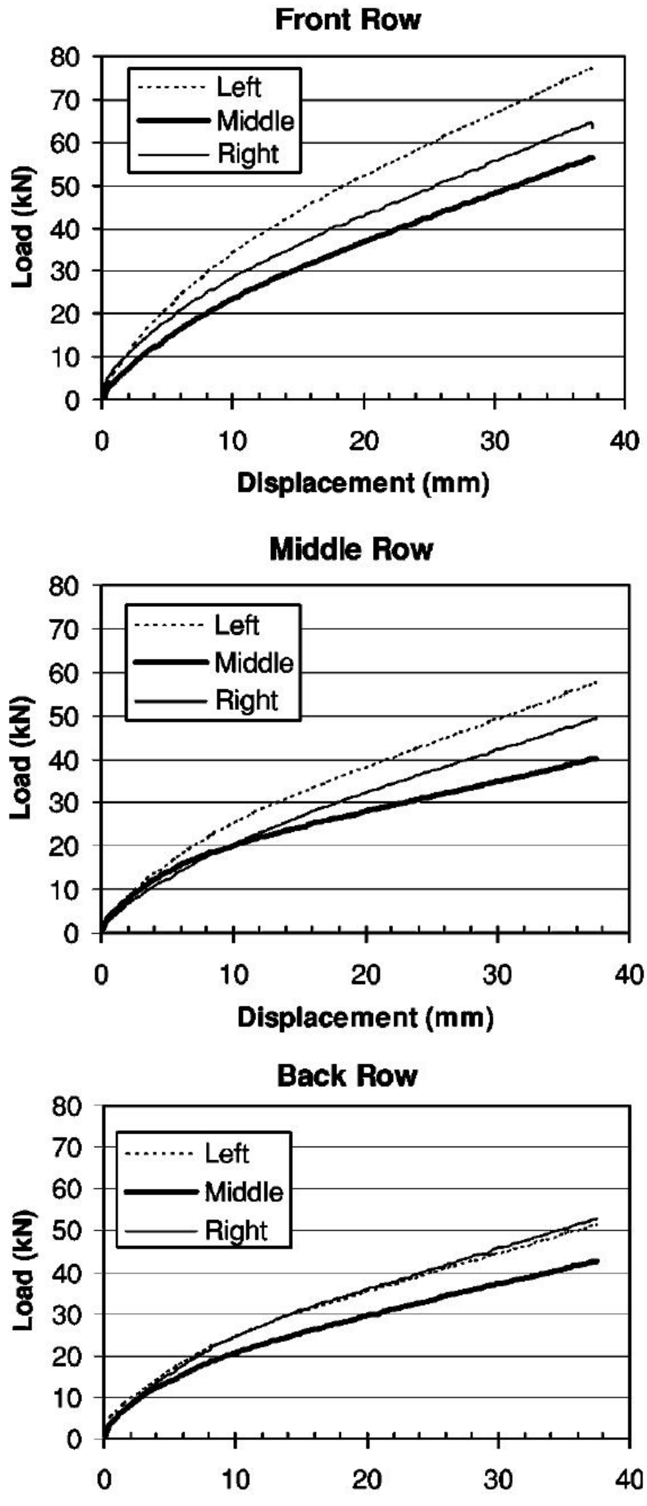


Figure 2.5. Measured load – deflection curves for left, middle and right piles in each row, Rollins (2001)

In order to check the predictive capability of the methods proposed in the literature, Zhang and McVay (1999) conducted a research on groups with 3x3 to 3x7 piles. Model piles made up of aluminum were tested in the centrifuge at 45g. Both the single pile and group testing were conducted and it was concluded that the method proposed by Reese et al. (1974) was suitable for the calculation of lateral response of a single pile in sand. It was also stated that the approach suggested by Brown et al. (1988) for determination of p – multipliers was simple but useful for the characterization of shadowing effects in group of piles. Another research, similar to the previous was conducted by Hamilton and Dunnavant (1993) on the results of the tests at Houston. This research was focused on the importance of the selection of soil parameters in the analytical methods. For example the method of Reese et al. (1974) may result in quite different values when compared with the test results.

An interesting conclusion was drawn from the test results of Patra & Pise (2001) who studied a number of pile groups with different arrangement of piles. It was reported that the pile groups having rough piles provided greater lateral resistance than groups with smooth piles.

Tests on prototype piles of 0.43 m diameter by McVay (1995) aimed at calculation of the group efficiency factors. Two different tests were performed including pile groups of 3 diameter and 5 diameter spacing. Methods of Reese et al. (1974) and Brown et al. (1988) as illustrated in Figure 2.6 were recommended for the analysis of single pile in sand. A more important result the authors reached was related to the group efficiencies. For the group spaced at 3 diameters, the total group efficiency was found to be 0.74 for most displacement values. For the group of piles spaced at 5 diameters, the efficiency increases to 0.93 for most displacements as would be expected.

Mokwa and Duncan (2001) reported that the pile cap influenced the lateral behavior of a pile group. A total of 31 tests were performed on full–scale piles and for soil type used in the study (hard sandy clay), the pile caps provided about 50% of the lateral resistance of the total system. Based on the test results, the stiffness and the strength of soil in front of the cap are primarily effective on response. Also, the depth and the size of the pile cap should be selected according to the required lateral resistance.

The list of the relevant studies on the subject is long, but many of them yielded similar results with those cited in this section.

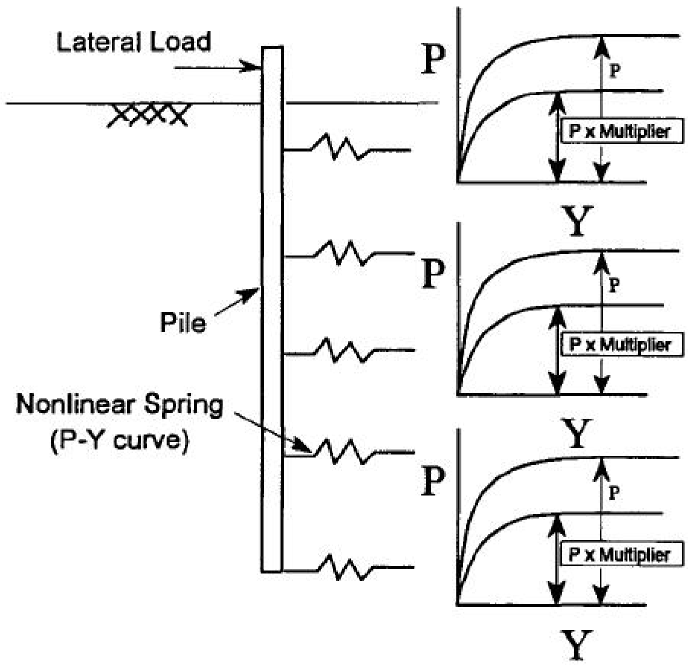


Figure 2.6. P-y multiplier approach for individual rows.
(Brown et al., 1988)

2.2 Recommendations of Design Specifications

Many specifications throughout the world include a distinct section on the design of laterally loaded piles and pile groups. Some of these widely used specifications and the respective recommendations given are summarized in the following.

2.2.1 US Army Corps of Engineers EM 1110-1-1905

As stated in this manual, failure of the pile is commonly due to exceedence of the soil bearing capacity in the case of short piles and drilled shafts which behave as rigid members. On the other hand, long piles are critical when the excessive pile deflection and high bending moments are concerned. Also, the loading conditions are said to play an important role in the lateral behavior of the pile. For cyclic loading case, engineer is warned against possible reduction in the soil resistance, formation of gaps around the shaft, and the corresponding increase of lateral deflection.

2.2.2 AASHTO LRFD Bridge Specifications

According to section 10.7.3.11 of this specification, lateral resistance of single piles should be reduced by 25% for cohesionless soils and 15% for cohesive soils in assessing the lateral capacity of pile groups. Single pile resistance can be calculated using the method of Reese. However, there is no information regarding cyclic loading. In the section A10.2 (foundation design), lateral pile behavior is explained in greater detail. It is stated that a span of 5 pile diameters from ground surface is usually the most effective part of a lateral load bearing pile. Also the methods of the specification prepared by American Petroleum Institute for offshore platforms is suggested for analysis.

2.2.3 AASHTO Standard Specifications for Highway Bridges

In section 4.6.5.6 of the specification, it is stated that lateral movement criteria is the most important check for the design of laterally loaded piles and in the design of laterally loaded piles it should be noted that there will be soil-structure interaction between the pile and ground. For example, Reese account for this interaction in the calculations. In order to select the pile sections for preliminary design process, ultimate lateral capacity or deflection of laterally loaded piles can be found by several methods (e.g. Broms, 1964a and 1964b; Singh et. al., 1971). Also the main factors affecting the lateral pile capacity is listed as the layering of soil, the elevation of ground water, the possibility of scour and the group action between the piles. For the group action, it is said that there is no reliable method for calculation of the group efficiency and suggested ratios of center–to–center spacing are summarized in Table 2.3. For design of piles against cyclic loading, specification recommends the use of COM624 analysis by Reese (1984).

2.2.4 API (American Petroleum Institute) Offshore Platforms Specification

Lateral pile behavior is explained in a more detailed fashion in section 6.8 of this specification and the required equations for analysis are given. The equations are given for soft clay, stiff clay and sand separately with the corrections for cyclic

Table 2.3. Lateral capacity reduction factors for the piles in a group according to CTC spacing (CGS ,1985)

Center – to – center shaft spacing for in – line loading	Ratio of lateral resistance of shaft in group to single shaft
8B	1.00
6B	0.70
4B	0.40
3B	0.25

loading conditions. For the geotechnical analysis, research done by Matlock (1970), Reese (1975) and O'Neill (1983) are used. The specification also states that the mathematical model may not always reflect the pile behavior realistically as these methods are approximate and that the results are mostly useful for analysis but sometimes unexpected results may possibly come out depending on the circumstances.

2.2.5 Eurocode

According to European Union Specification, EN1992-1-1:2004 (E) section G2, except from lateral loads, axial loads along the pile should also be considered for the determination of lateral capacity. However, no reference or equation is provided to define this axial and lateral load relationship.

2.2.6 <u>Arema</u>

In section 8-24.3.2.2 of the specification it is stated that the soil behavior should be determined to define the behavior of laterally loaded piles but no reference or detailed explanation is given to support this statement.

2.2.7 <u>Caltrans</u>

As mentioned in the section 7.7.1.2.2 of California Highways Specification, apart from shear force, bending moment, axial force and rigidity of the pile, lateral capacity and the stability of the soil should also be known for the design of piles against lateral loads. In addition, it is stated that the positive effect of the pile cap to the lateral resistance can be only considered in the active state. In this specification, there is no recommended reference or method for the calculation of lateral pile capacity. However, reduction due to group effect is not recommended in contrast to AASHTO Standard Specifications for Highway Bridges.

2.2.8 ATC 32 (Applied Technology Council)

In the book 32 of Applied Technology Council, it is said that the methods of Reese (1984) and Matlock (1970) are suitable for the determination of lateral pile behavior. For the modeling of the pile under lateral loads, beam members are to be used and the soil response is defined by nonlinear springs. In this book, an alternative method called equivalent diameter method is also stated and relevant references are given.

2.2.9 ACI (American Concrete Institute)

Specification No 543 of American Concrete Institute requires the design of piles against lateral loads in Section 2.1.9.2 but no information is available how this design can be done.

2.3 General Aspects of Lateral Pile Response

For preliminary design process, linear springs are used to model the soil response. Especially in sands, the lateral soil resistance can be properly modeled by using elastic, closely spaced springs at required depths. However, for final design, this method is disadvantageous since the modulus of subgrade reaction is not a unique soil property as illustrated in Figure 2.7 and Figure 2.8.

Not only for high loading levels but also for the low range, there is a nonlinear relationship between the load, deflection and moment in piles. The two factors defining the response of piles under lateral loading are:

1) Nonlinearity of the load – deflection behavior of soil around the pile. During lateral loading, the soil resistance increases with increasing lateral loads. The deflection, however, increases more rapidly.

2) When the soil strength at the upper part of the pile is mobilized, loads are transferred to greater depths which results in the increase of span length. This increase becomes more rapid in bending since both load and moment arm is increased.

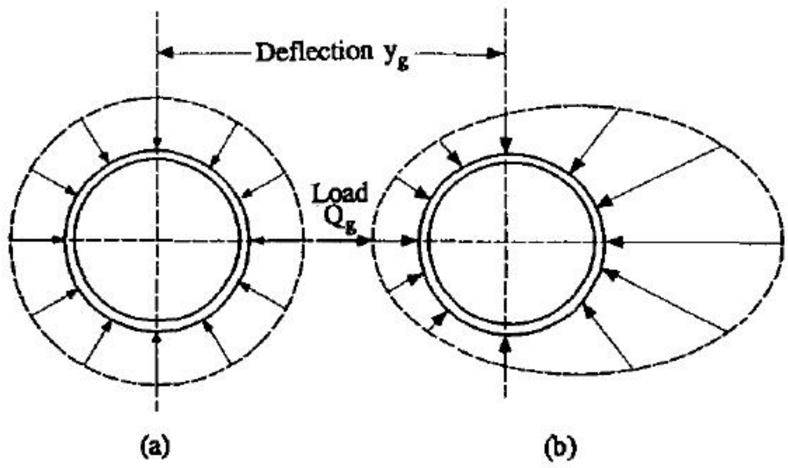


Figure 2.7. Distribution of soil reaction around a pile: a) before lateral load; b) after lateral load (Prakash & Kumar, 1996)

P-y analyses can be used for different soil and loading conditions and the results of these analyses are in good agreement with the full scale field loading tests. The disadvantage of using p-y analyses, however, is the duration of time required to create the input and the detailed computer analyses. Accordingly, p-y analyses are generally conducted for major projects. For the specific case of bridge piers, the main sources of lateral loads are ship impact, wind or earthquake. In modelling of a bridge, superstructure is usually modeled together with piles where

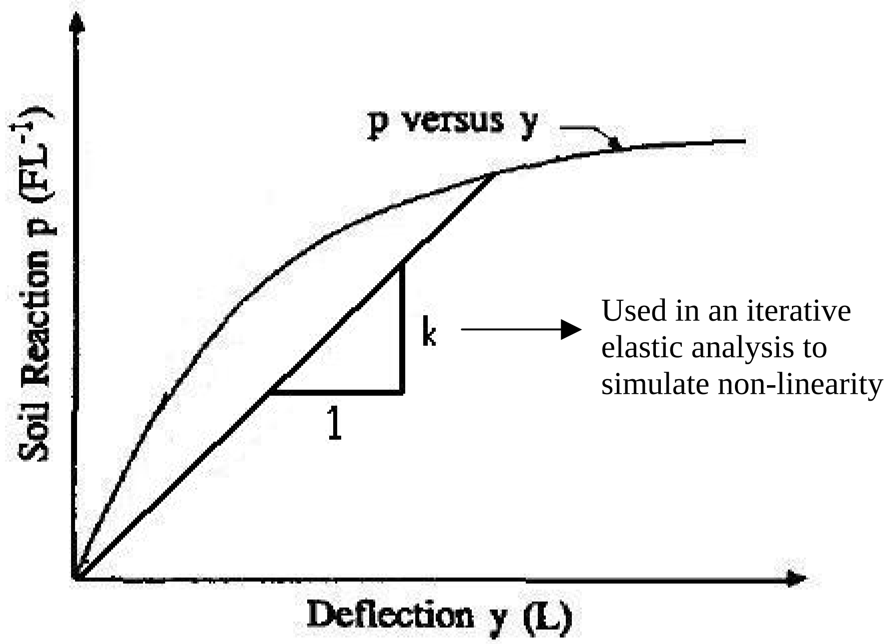


Figure 2.8. Nonlinear behavior of soil (Prakash & Kumar, 1996)

piles are idealized as unsupported columns of some fixed length. This point of fixity can be found by a beam-column analysis performed by using nonlinear springs representing the soil. There exist recommendations for the depth that will be the effective in the lateral response of piles. Most commonly used procedure states that when the lateral response of a pile is considered, the soil around the top of the pile or drilled shaft is the most critical part. As can be seen from the test results of Huang (2001), conducted on 1.5 m diameter bored piles and 0.8 m diameter driven piles (Figure 2.9), the deflection of pile approaches to zero approximately at a depth of 8 pile diameters. Also, the response is subject to change when the pile or drilled shaft cracks. For these cases the flexural stiffness of the pile should be appropriately reduced.

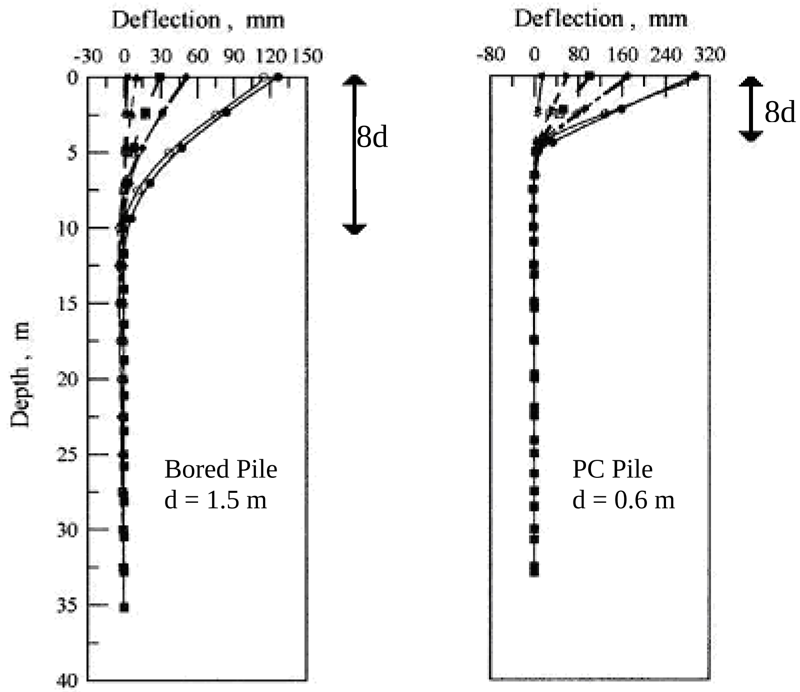


Figure 2.9. Depth of fixity depending on the pile diameter (Huang, 2001)

Computer modelling of laterally loaded piles is typically done by utilizing finite-difference models with nonlinear springs. Softwares such as LPILE, GROUP or general purpose structural analysis programs treat the pile as a beam and the soil as nonlinear springs. Springs are typically assigned at every 1 m depth in industry practice for a detailed description of the soil resistance throughout the pile length. In their study Prakash and Kumar (1996) reported that lateral load deflection of piles in sands could be analyzed by considering the pile as a beam on elastic foundation. In this case, soil was replaced by closely spaced elastic springs. This method was relatively simple for modelling but the modulus of subgrade reaction modulus is not a unique property. Alternatively, Rollins (2005) assigned p-y curves throughout the pile length for an analysis in LPILE. Load-deflection characteristics of test results and LPILE model were close to each other.

Once the lateral load – deflection characteristics for a single pile is identified, the next step in design process is the determination of the behavior of piles in a group. Pile head boundary conditions become important for determination of p-multipliers. For pile groups, piles are mostly fixed head. Also, pile spacing is important. When piles are widely spaced, total response of the group will be equal to the summation of individual response of all piles. When piles are closely spaced, behavior of a single pile is influenced through the adjacent soil by the response of other piles nearby. This effect is, usually named as “shadowing effect”. Shadowing effect was studied by several researchers through lateral load tests on pile groups as summarized in Table 2.4. Davisson (1970) claimed that there was no pile-soil-pile interaction when pile spacing is more than eight diameters in the direction of loading. On the other hand, for a center to center spacing of three diameters, modulus of subgrade reaction might be reduced to 25% of its original value in order to include the effect of pile-soil-pile interaction. A different study by Arsoy and Prakash (2001) stated that group action disappears at 6 diameter spacing for 2x2 group and 7 diameter for groups having 6 piles or less in the direction of loading. Therefore, the behavior of pile group may be determined from the response of a single pile which is calculated by using the reduced modulus. For pile groups with pile spacing between three and eight diameters, linear interpolation can be done for the reduction of subgrade reaction. In contrast, ATC-32 and Caltrans recommended that the group effect could be neglected for earthquake loading at three center to center spacing or higher since static load tests are said to overestimate group effect.

Moreover, there are some other circumstances affecting lateral performance of the pile groups such as the lateral resistance provided by the pile cap, the method of installation and the moments applied at the ground surface. When the contribution of pile cap is considered, two contradictory views are present in practice. First, following the construction stage, pile cap remains in contact with the ground, which provides lateral resistance to the pile group. Based on the full – scale

Table 2.4. Summary of reduction factors based on full scale and laboratory model tests.

Researcher	Test Type	Pile Group	Pile Fixity	Soil Type	Center to center spacing	Shear Strength Parameters	1^{st} row	2^{nd} row	3^{rd} row	4^{th} row	5^{th} row	6^{th} row	7^{th} row
Meimon et al. (1986)	Full Scale	3x2	Free Head	Clay	3D	s_u = 20 kPa	0.9	0.5	-	-	-	-	-
Brown et al. (1987)	Full Scale	3x3	Free Head	Clay	3D	s_u = 72 kPa	0.7	0.6	0.5	-	-	-	-
Rollins et al. (1998)	Full Scale	3x3	Free Head	Clay	3D	s_u = 48 kPa	0.6	0.4	0.4	-	-	-	-
Sparks and Rollins (1987)	Full Scale	3x3	Fixed Head	Clay	3D	s_u = 48 kPa	0.6	0.4	0.4	-	-	-	-
Brown et al. (1988)	Full Scale	3x3	Free Head	Sand	3D	$\phi = 38°$	0.8	0.4	0.3	-	-	-	-
Brown et al. (2001)	Full Scale	2x3	Fixed Head	Sand	3D	$\phi = 35°$	0.5	0.4	0.3	-	-	-	-
Brown et al. (2001)	Full Scale	3x4	Fixed Head	Sand	3D	$\phi = 35°$	0.9	0.7	0.5	0.4	-	-	-
Ruesta & Townsend(1997)	Full Scale	4x4	Free Head	Sand	3D	$\phi = 32°$	0.8	0.7	0.3	0.3	-	-	-
McVay et al. (1998)	Centrifuge	3x3	Free Head	Sand	5D	D_r = 33%	1.0	0.9	0.7	-	-	-	-

Table 2.4. (continued)

Researcher	Test Type	Pile Group	Pile Fixity	Soil Type	Center to center spacing	Shear Strength Parameters	1^{st} row	2^{nd} row	3^{rd} row	4^{th} row	5^{th} row	6^{th} row	7^{th} row
McVay et al. (1998)	Centrifuge	3x3	Free Head	Sand	3D	D_r = 55%	0.8	0.4	0.3	-	-	-	-
McVay et al. (1998)	Centrifuge	3x3	Free Head	Sand	5D	D_r = 55%	1	0.9	0.7	-	-	-	-
McVay et al. (1998)	Centrifuge	3x3	Free Head	Sand	3D	D_r = 36% and 55%	0.8	0.4	0.3	-	-	-	-
McVay et al. (1998)	Centrifuge	3x4	Free Head	Sand	3D	D_r = 36% and 55%	0.8	0.4	0.3	0.3	-	-	-
McVay et al. (1998)	Centrifuge	3x5	Free Head	Sand	3D	D_r = 36% and 55%	0.8	0.4	0.3	0.2	0.3	-	-
McVay et al. (1998)	Centrifuge	3x6	Free Head	Sand	3D	D_r = 36% and 55%	0.8	0.4	0.3	0.2	0.2	0.3	-
McVay et al. (1998)	Centrifuge	3x7	Free Head	Sand	3D	D_r = 36% and 55%	0.8	0.4	0.3	0.2	0.2	0.2	0.3
Remaud et al. (1998)	Centrifuge	1x2	-	Sand	2D	Dense	1	0.5	-	-	-	-	-
Rollins et al. (2005)	Full Scale	3x3	Free Head	Sand	3.29D	$\phi = 32°$	0.8	0.4	0.4	-	-	-	-
Huang et al. (2001)	Full Scale	3x4	Fixed Head	Sand	3D	D_r = 40% ~ 60%	0.9	0.6	0.6	0.7	-	-	-

test results, this contribution may be as high as 50% of the total lateral resistance, which cannot be ignored in the design process. Alternatively, often in design the lateral resistance provided by the pile cap is ignored to remain on the safe side.

Effect of installation is another factor on the performance of the pile group. For example, during the installation of driven piles, the soil around the pile becomes stiffer. On the other hand, bored piles can result in different lateral resistance according to the use of casing. For the bored piles where casing is used, the pile surface will be smooth whereas for no casing the surface will be rough. The roughness of the surface provides an extra lateral resistance. However, none of the present analysis methods include adjustments for installation effects since such effects need still to be investigated further.

When a lateral load is applied above ground line, moment is introduced to the pile as well. Since the behavior of pile-soil system is nonlinear, the deflections due to moment and lateral load cannot be superposed directly. In order to overcome this problem, a nonlinear superposition procedure can be carried out. For the determination of total lateral deflection, first the lateral movements caused by lateral load and moment are determined separately as shown in Figure 2.10(a,b). Then, a value of load, called equivalent load, that would cause the same deformation as the moment is found (Figure 2.10(c)). Similarly, a value of moment, called equivalent moment, causing the same deformation as the load is calculated (Figure 2.10(d)). Next, the deformations are calculated caused by the actual and equivalent loads as well as real and equivalent moments (Figure 2.10(e,f)). Total ground line deflection is the average of these two values.

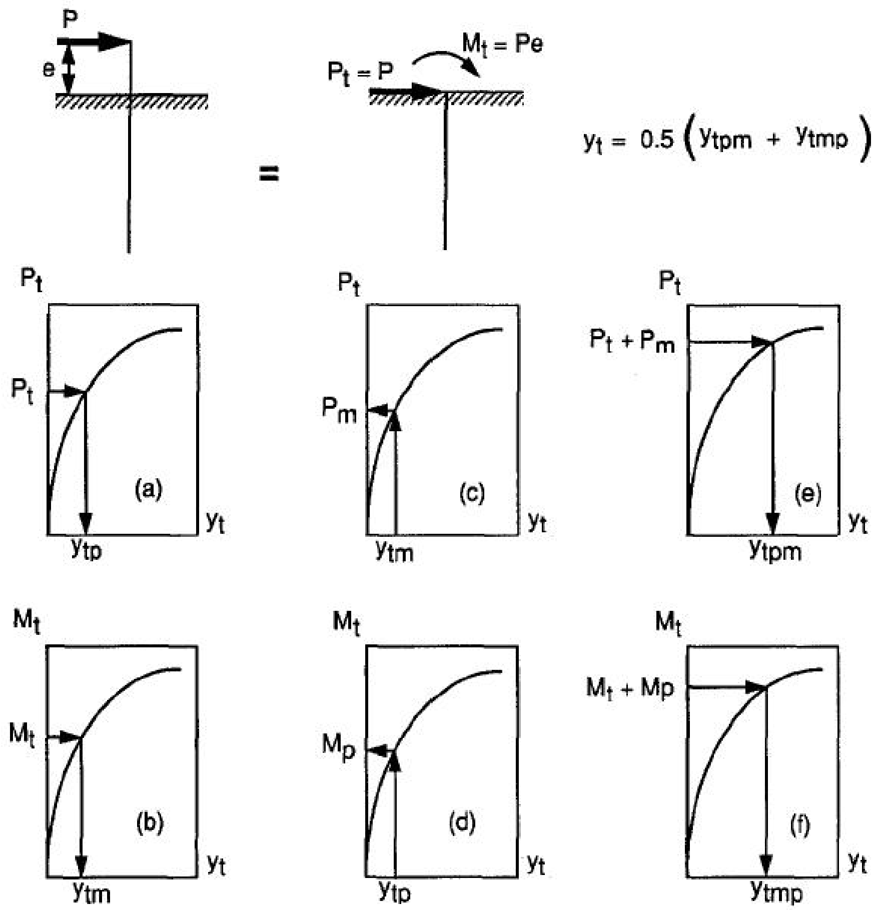


Figure 2.10. Nonlinear superposition procedure for a load applied above ground (Duncan et al., 1994)

CHAPTER 3

METHOD OF ANALYSIS

3.1 Introduction

As summarized in Chapter 2, almost all researchers concluded that pile–soil interaction is nonlinear. A great majority of the methods presented in literature consider this nonlinearity. There exist widely used software such as LPILE, Allpile, FLPier, etc. that can be used in nonlinear solutions. These softwares consist of their own libraries where parameters for different soil types are assigned. In the case of lack of data for input, the program can assign a suitable representative value from the library. On the other hand, the capability of graphical output for deflection, bending moment and soil resistance at any required depth along the pile makes these softwares a useful tool.

In Turkey, for a great majority of projects, geotechnical investigations include only the Standard Penetration Test (SPT) data and rather limited laboratory test results from disturbed soil samples. Boring logs from test sites include SPT-N values, soil classification based on visual inspection, and the level of groundwater. In some cases, samples are retrieved and laboratory tests such as sieve analysis, Atterberg limits and unconfined compression are performed. For fine grained soils, these tests can be useful since they provide information regarding plasticity and cohesion. However, for coarse grained soils, undisturbed sampling for laboratory testing is so difficult. Accordingly, in Turkey, the correlations between SPT-N values and soil parameters are commonly used.

Usually, lack of information on soil parameters, makes the use of available sophisticated software obsolete. Moreover, the time inefficiency and complexity of nonlinear hand calculation procedures increase the popularity of elastic solution. In linear analysis, horizontal subgrade reaction modulus is an important parameter influencing the design. Pile length and reinforcement are determined based on the results of linear analysis, usually resulting in overdesign due to the high factor of safety involved in the selection of modulus of subgrade reaction. In order to overcome the overdesign, nonlinearity is to be introduced into the analyses. A simple formulation can be a useful tool for the selection of initial dimensions and range of reinforcement ratio for a pile.

In Turkey, bored piles are widely preferred with diameters ranging from 60 cm to 200 cm. In some specific projects, precast driven piles, steel pipe piles or micropiles are used but their application is rather limited. The procedure that will be described in detail in the following pages will be applicable for concrete piles only.

3.2 Step-by-Step Design Procedure

A pile in a foundation system can be detached from the structure and analyzed as a single member. In this case, the boundary conditions should be properly considered such as the fixity of the pile head or the resistance of soil in front of the pile. The behavior of a pile under lateral loads can be simulated assuming a point of fixity at some depth along the pile and a distributed lateral force representing the soil resistance against a point load and/or moment. The first parameter that will be obtained from such a system is lateral deflection. The basic concepts of structural analysis can be utilized to determine the pile displacements.

An important point at this stage is the selection of moment of inertia. When the moment in any reinforced concrete member reaches at a specific value, the section cracks, resulting in a reduction in the moment of inertia of the member. In order to calculate the displacements at the top of the pile, a reduced value referred as "effective moment of inertia" should be used. In section 9.5.2.3 of ACI 318

Building Code, this concept is summarized and also using an effective inertia value "I_e" which is less than the gross moment of inertia "I_g" is suggested. For the calculation I_e, following formulation may be used.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad \text{(Equation 3.1)}$$

In this formula M_{cr} is the value of moment at which the concrete section is cracked and can be calculated as:

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad \text{(Equation 3.2)}$$

where; $f_r = 7.5\sqrt{f_c'}$ (Equation 3.3)

I_g : Gross moment of inertia of the section (= $\pi D^4/64$ for circular piles).
y_t : Distance from the centroid of the section to the extreme fiber.
M_a : Fixed end moment created by the lateral load.

Cracking introduces additional complexity into the analysis. For the case of laterally loaded piles, the depth of fixity is dependent on the effective moment of inertia of the pile section. Similarly, the fixed end moment in the formulation of I_e depends on the depth of fixity. In order to solve both of the equations simultaneously, an iterative procedure is required. In the absence of a computer code, the effective moment of inertia can be taken as the half of the gross moment of inertia for simplicity in the calculations.

3.2.1 Data Used In Back Calculation Process

Due to the fact that reinforced concrete piles are the most commonly used deep foundation type in Turkey, the simple method in this study is developed for such piles. The empirical data used in the analysis has been collected from literature

Table 3.1. Selected test data for method development

Test Site	Researcher	Pile Diameter (m)	Pile Length (m)	Pile System	Mod. Of Elast. Of Concrete (GPa)	Pile Head Condition	Applied Load (kN)	Applied Moment (kN.m)	Measured Deflection (mm)
HOUSTON	Reese & Welch (1975)	0.762	12.8	Single Pile	24.2	Free Head	450	34.2	29.58
							360	27.36	14.63
							270	20.52	6.13
							180	13.68	2.13
							90	6.84	0.91
PORTO TOLLE	Jamiolkowski &Garrasino (1977)	0.5	45	Group (3x1)	21.4	Fixed Head	441	0	8
HONG KONG	Ng et. al. (2001)	1.5	21	Group (2x1)	32.3	Fixed Head	2875	0	69
							2500	0	43
							1975	0	25.5
							1450	0	14.5
							950	0	4.5
							500	0	2
TAIWAN	Huang et. al. (2001)	1.5	34.9	Group (2x3)	24.7	Fixed Head	563	0	3.45
							1057	0	10.89
							1372	0	16.94
							1580	0	23.4
							1806	0	29.36
TRABZON	MNG ZEMTAŞ A.Ş. (2002)	1	4	Single Pile	20	Free Head	200	50	2
							150	37.5	1.17
							100	25	0.43
							50	12.5	0.17

Table 3.1. (continued)

Test Site	Researcher	Pile Diameter (m)	Pile Length (m)	Pile System	Mod. Of Elast. Of Concrete (GPa)	Pile Head Condition	Applied Load (kN)	Applied Moment (kN.m)	Measured Deflection (mm)
LAS VEGAS	Zafir & Vanderpool (1998)	0.61	7	Group(2x2)	32	Fixed Head	813	0	24.3
							749	0	20.79
							696	0	17.16
							645	0	14.11
							594	0	11.12
							534	0	9.07
							485	0	7.23
							426	0	5.61
							372	0	4.42
							318	0	3.15
							270	0	2.32
							211	0	1.48
							156	0	1.15
							101	0	0.53
SAKHALIN	Düzceer (2004)	0.6	N/A	Single Pile	20	Free Head	300	0	39.4
							200	0	10.3
							100	0	4.61
MERSİN	Toker Sondaj ve İnş. A.Ş. (2004)	0.8	22	Single Pile	20	Free Head	77.5	0	0.54
							153.6	0	1.7
ÇATALAĞZI	Toker Sondaj ve İnş. A.Ş. (2001)	0.65	20	Single Pile	20	Free Head	75.5	0	0.65
							159.4	0	3.33
							188	0	5.84
							225.8	0	8.38
							263.7	0	12.12

Table 3.1. (continued)

Test Site	Researcher	Pile Diameter (m)	Pile Length (m)	Pile System	Mod. Of Elast. Of Concrete (GPa)	Pile Head Condition	Applied Load (kN)	Applied Moment (kN.m)	Measured Deflection (mm)
GARSTON	Price & Wardle (1987)	1.5	12.5	Single Pile	33.2	Free Head	216	194.4	0.82
							433	389.7	3.05
							798	718.2	8.73
							1000	900	12.82
							1226	1103.4	19.18
							1400	1260	25.09
							1596	1436.4	31.59
							1784	1605.6	40
							2000	1800	46.41
							2163	1946.7	56.36
							2356	2120.4	63.18
N/A	Bhushan & Lee (1981)	0.6	N/A	Single Pile	Not Reported (Assumed as 27.8)	Free Head	105	0	1.91
							217	0	4.65
							327	0	8.22
							439	0	14.15
							550	0	20.58
							653	0	28.06
							887	0	39.77
N/A	Ismael & Klym (1978)	1.5	11.6	Single Pile	28	Free Head	618	0	9

Table 3.1. (continued)

Test Site	Researcher	Pile Diameter (m)	Pile Length (m)	Pile System	Mod. Of Elast. Of Concrete (GPa)	Pile Head Condition	Applied Load (kN)	Applied Moment (kN.m)	Measured Deflection (mm)
TAIWAN	Huang et. al. (2001)	1.5	34.9	Single Pile	24.7	Free Head	408	0	4.81
							723	0	11.86
							1116	0	19.15
							1455	0	25.96
							1614	0	33.25
							1762	0	38.46
							1843	0	42.87
							1891	0	49.76
							2039	0	56.49
							2386	0	73
							2549	0	86.38
							2674	0	98
							2925	0	118.59

and summarized in Table 3.1. In the table, dimensions of the pile, modulus of elasticity of concrete, soil type and the measured load –deflection values are given. In all sources, the load versus deflection data is given in the graphical form and for completeness this graphical information is provided in Appendix B.

3.2.2 Proposed Simple Method

In this section a simple method that can be used to determine the pile deflection and bending moment will be presented. The method is based on a formulation which is derived from the back analysis of the available test results. The formulas describing the deflection of a pile under lateral loads are given in Equations 3.4 and 3.5. Those equations originate from beam deflection formulas as cited by Hibbeler. For simplicity of the proposed method, deflections due to lateral load and moment are linearly superposed rather than using a nonlinear superposition procedure. Factors in the formulas are empirical constants determined using the available data and may show slight difference when applied to different test results.

$$\delta = \left(\frac{P\alpha^3 D^3}{3EI_e} + \frac{M\alpha^2 D^2}{2EI_e} \right) \frac{\beta}{\eta} \quad \text{for free head piles} \qquad \text{(Equation 3.4)}$$

$$\delta = \left(\frac{P\alpha^3 D^3}{12EI_e} + \frac{M\alpha^2 D^2}{6EI_e} \right) \frac{\beta}{\eta} \quad \text{for fixed head piles} \qquad \text{(Equation 3.5)}$$

where;

P : Applied lateral load at top of pile

M : Applied moment at top of pile

D : Diameter of pile

E : Modulus of elasticity of concrete of pile

I_e : Effective moment of inertia of pile cross section

α : Empirical constant for depth of fixity

β : Empirical constant for soil type

η : Empirical constant for group efficiency

3.2.2.1 Determination of the factor for depth of fixity (α)

When a lateral load or a moment is applied, the pile will deflect in response, and a point called as "depth of fixity" will occur. Depth of fixity can be related to the lateral load applied on the pile. In reality occurrence of this point is also affected from soil resistance and pile group reduction. These additional factors of influence will be included in the analysis following calculation of depth of fixity. In the first part of the analysis the pile length from top of the pile to the point of fixity is thought as a cantilever member which is fixed at the bottom end for free head piles, and fixed at both ends for the case of fixed head piles. The assumption in the analysis is that there is no soil resisting in front of the pile and there is no group reduction due to the neighboring piles. Therefore, the pile can be solved for the depth of fixity as a cantilever column with no lateral resistance. For the calculation of length of theoretical cantilever member, deflection, modulus of elasticity and effective moment of inertia values are required. Deflection of the member can be calculated as;

$$\delta = \left(\frac{PL^3}{3EI_e} + \frac{ML^2}{2EI_e} \right) \quad \text{for free head case} \qquad \text{(Equation 3.6)}$$

$$\delta = \left(\frac{PL^3}{12EI_e} + \frac{ML^2}{6EI_e} \right) \quad \text{for fixed head case} \qquad \text{(Equation 3.7)}$$

E and I_e values are dependent on the material and dimensions of the section. In these formulas the only unknown is the length from top of the pile to the point of fixity "L". A relationship between the depth of fixity and lateral load can be established by performing a back calculation of the formula using the full – scale test load versus deflection values. At this point, an empirical constant is introduced which correlates the span length and pile diameter. As some researchers indicated, lateral pile behavior is determined along a depth of 5 to 8 times the diameter of the pile. Placing an "α" factor to the analysis is a similar concept with 5 to 8 diameter recommendation. For a specific diameter of the pile, α becomes the only unknown.

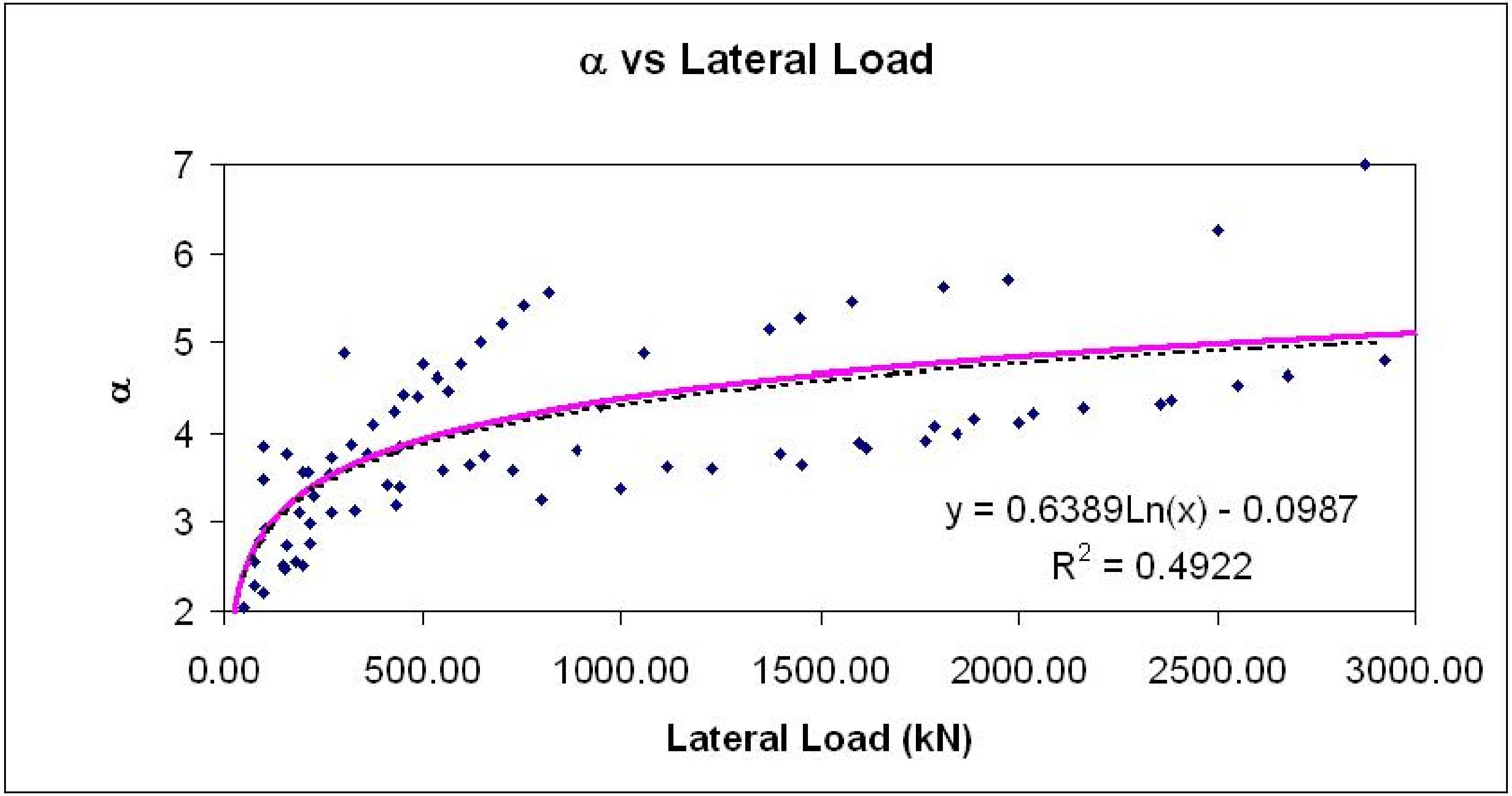


Figure 3.1. Relationship between lateral load and depth of fixity factor (α) for I_e calculated by ACI 318

When α versus lateral load is plotted, a logarithmic relationship is observed to exist, as shown in Figure 3.1. The dashed line represents the best fit curve with R^2 value of 0.49. This value may seem to be low for establishing a relationship but it should be remembered that it is for the case where no soil resistance is present.

The continuous line is the plot of the following equation:

$$\alpha = 0.65\ln(P) - 0.10 \qquad \text{(Equation 3.8)}$$

Above equation is for effective moment of inertia values which are calculated according to the ACI 318 Code by the help of a computer program. For simplicity, effective moment of inertia can be taken as half of the gross moment of inertia. When the same test results are analyzed by placing $0.5I_g$ for I_e in the formula, a slight change in the distribution of data points occur as shown in Figure 3.2. Equation 3.8 is also plotted in Figure 3.2 in order to check its suitability for further calculations. Looking at Figure 3.1 and Figure 3.2, the proposed equation appears to be suitable for both cases. For a numerical check, a lateral load value of 500 kN is selected and the equations are solved for α.

- For best logarithmic relation of Figure 3.1:

$$\alpha = 0.6389 \times \ln(500) - 0.0987 = 3.8718$$

- For best logarithmic relation of Figure 3.2:

$$\alpha = 0.7284 \times \ln(500) - 0.7672 = 3.7595$$

- For Equation 3.8:

$$\alpha = 0.65 \times \ln(500) - 0.10 = 3.9394$$

According to the results presented above, the greatest difference between α values is about 0.18. For a pile with a diameter of 1 m, the point of fixity will be directly equal to α values. Using the sample calculation shown above, for a 1 m diameter pile the maximum difference between three methods comes out to be 18 cm, which is tolerable. As a result, no matter how the effective moment of inertia is calculated, the relationship between the lateral load and depth of fixity can be defined by Equation 3.8.

3.2.2.2 Determination of the factors for soil type (β) and group efficiency (η)

In the previous section, a relationship is given for the depth of fixity of a pile under a lateral load neglecting the effect of soil resistance. Naturally, that relation does not reflect the real behavior except for very soft soils for which the lateral soil resistance is negligible. Based on the test results available in literature, the lateral behavior of a pile is different for different soil types.

Here, a factor for the contribution of soil will be introduced to the equation of deflection. Logically, this factor should be less than unity, which is representative of the case of absence of soil and should be greater than zero representing infinitely stiff soil case. Computation of β factors includes another back calculation of the test results, by using α value, which is determined from Equation 3.8. In order to make β the only unknown in Equation 3.4 or Equation 3.5, the group efficiency is neglected. Assigning $\eta = 1.0$, β factors are found from the available load – deflection data. For each load value, a different α is calculated resulting in a different β. However, it will be reasonable to average those β values among the tests performed in the same soil type to obtain a representative factor. This calculation is done by using a simple computer code and summarized in Table 3.2.

As it can be seen in the table that some β factors are larger than 1.0, which cannot theoretically be possible. The explanation for this inconvenience is the group reduction effects for piles. When the available test data is classified according to the pile condition (single or group), except for the test site at Sakhalin, all the β factors for single pile tests are lower than 1.0, as would be expected. Similarly, the β factors for group of piles except test site at Porto Tolle are larger than 1.0 due to the group reduction in the lateral response.

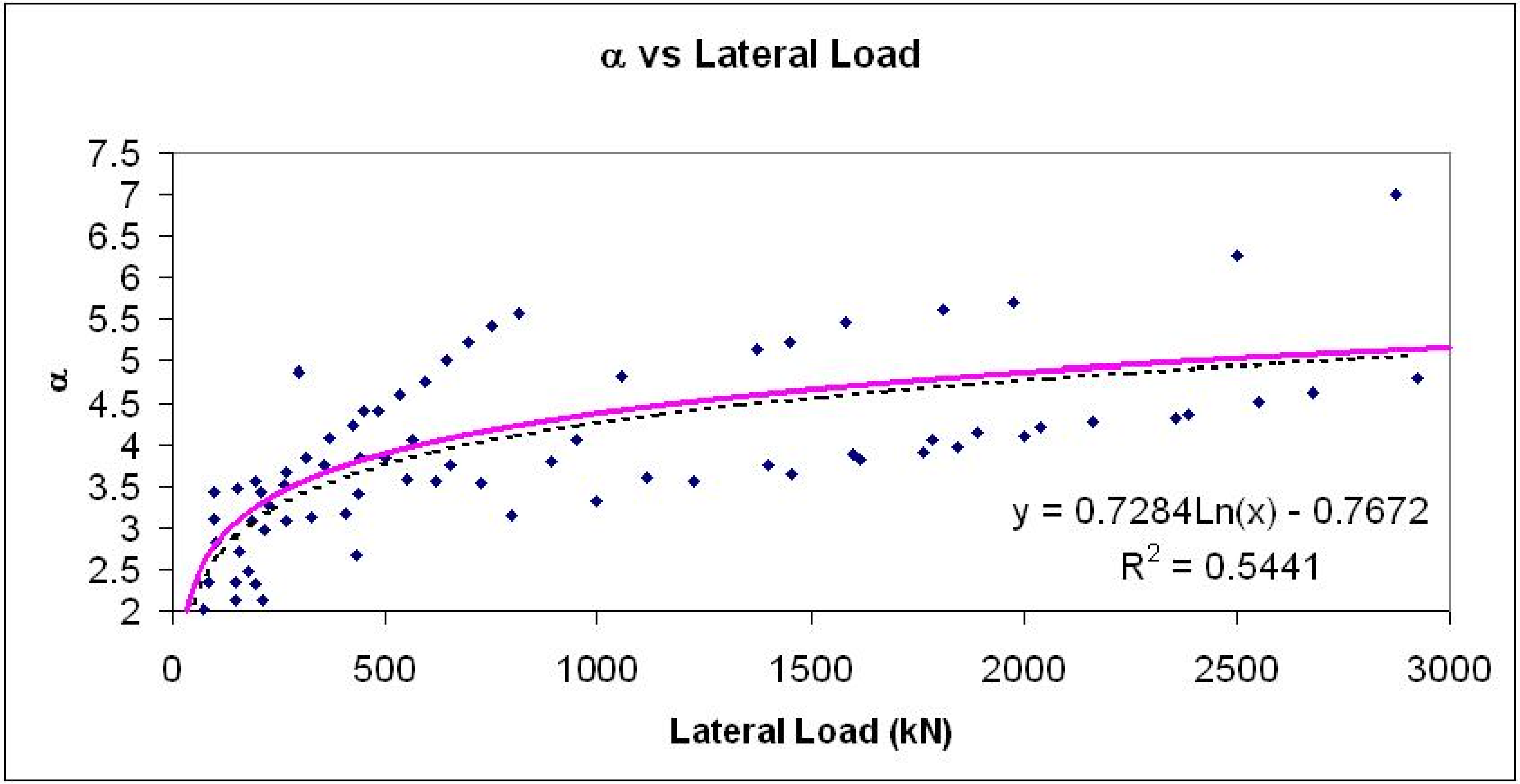


Figure 3.2. Relationship between lateral load and depth of fixity factor (α) for I_e taken as $0.5I_g$

For the group efficiency, p – multipliers were given in Table 3.4 for different test results. Tests performed by McVay et al. (1998) has shown that for a 3 diameter center – to – center spacing, average group efficiency of piles in sand for a 3x3 group is about 0.50 and for widely spaced piles such as 5 diameter spacing, this efficiency factor increases to 0.80. Here, these values are used in the analyses, and when the pile spacing is in between 3 to 5 diameters, linear interpolation is performed. Tests done by Meimon et al. (1986) and Brown et al. (1987) showed that for pile groups in clay, these reduction coefficients can be increased by specific amounts, such as 0.60 for 3 diameter spacing and 0.90 for 5 diameter spacing. For the determination of β factors, the group effect (η) should also be introduced to the equations for groups according to pile spacing. In Table 3.2, the back calculated β values for group of piles should be considered together with the effect of η value corresponding to the pile spacing.

3.2.2.3 Correlation between soil parameters and SPT-N values

In the following step, a correlation between β factors and SPT-N values will be established since particularly in Turkey, the most widely available field data is SPT-N values. Some other soil parameters can be correlated to SPT-N values.

For the case of sands the internal friction angle (ϕ) can be related to SPT-N blow counts by using the empirical equation below for round grains of uniform size proposed by Dunhum (1954).

$$\phi' = \sqrt{12N} + 15 \qquad \text{(Equation 3.9)}$$

For sands, the relationship between SPT-N values and relative density is proposed by Meyerhof (1956). (Table 3.3)

Table 3.2. Summary of soil type and back-calculated β values for the field tests (values in parentheses represent the β x η value for pile groups)

Test	**Soil Type**	**Available Soil Parameters**	36 **stem**	**Pile Head Condition**	**Back Calculated β Factor**	**Back Calculated SPT-N Value**
HOUSTON	Clay	c_u = 80 kPa ε_{50} = 0.005	Single Pile	Free Head	0.92	13
PORTO TOLLE	Sand	D_R = 40% ϕ = 35°	Group (3x1)	Fixed Head	0.72 (0.36)	33
HONG KONG	Sand	ϕ = 39°	Group (2x1)	Fixed Head	1.74 (1.39)	48
LAS VEGAS	Clay	c_u = 100 kPa e_{50} = 0.005	Group (2x2)	Fixed Head	1.64 (0.82)	16
TAIWAN	Sand	SPT-N = 15	Group (2x3)	Fixed Head	1.47 (0.74)	15
TRABZON	Clay	c_u = 80 kPa	Single Pile	Free Head	0.5	13
SAKHALIN	Silt	s_u = 65 kPa ϕ = 23°	Single Pile	Free Head	1.81	N/A
MERSİN	Gravelly Sand	N/A	Single Pile	Free Head	0.63	N/A
ÇATALAĞZI	Alluvium	N/A	Single Pile	Free Head	0.79	N/A
GARSTON	Sandy Gravel	SPT-N = 50 ϕ = 40°	Single Pile	Free Head	0.59	50
N/A	Clay	c_u = 50 kPa	Single Pile	Free Head	0.72	8
TAIWAN	Sand	SPT-N = 15	Single Pile	Free Head	0.65	15
N/A	Sand	N/A	Single Pile	Free Head	0.74	N/A

In the case of clayey soils, a relationship between SPT-N and cohesion (c_u) is needed. A correlation was given by Terzaghi and Peck (1953) for the calculation of unconfined compressive strength (q_u).

Table 3.3. Relative density versus SPT-N values (Meyerhof, 1956)

N - Value	Relative Density D_r (%)
<4	<20
4~10	20~40
10~30	40~60
30~50	60~80
>50	>80

$$q_u = \frac{N}{8} \quad \text{(Equation 3.10)}$$

$$q_u = 2c_u \quad \text{(Equation 3.11)}$$

Another back calculation is carried out for transforming the available soil parameters to SPT-N values.

Among the results of analyses, inconsistent cases appear such as Sakhalin and Hong Kong tests where β factor is greater than unity. This can be attributed to lack of information regarding the full-scale load tests. For some cases, only load versus lateral deflection data is provided. Except from the absence of soil parameters, the rotation that may take place in the pile cap is also an important factor since pile cap is assumed to be fixed in the proposed equations. A small amount of rotation may highly affect the reduction factor for group efficiency and that the back calculated β factor. The test from the Trabzon site is also inconsistent.

This is due to the rather short length of piles. Pile length is reported as 4 m, for which the pile under lateral load would tilt rather than bend. Therefore, the proposed equations are not applicable for such a pile. Table 3.4 contains the reduced data set following removal of the inconsistent data from the list.

Table 3.4. Remaining data after the removal of inconsistent test results

Test	Soil Type	Available Soil Parameters	Pile System	Pile Head Condition	Back Calculated β Factor	Back Calculated SPT-N Value
HOUSTON	Clay	c_u = 80 kPa ε_{50} = 0.005	Single Pile	Free Head	0.92	13
LAS VEGAS	Clay	c_u = 100 kPa e_{50} = 0.005	Group (2x2)	Fixed Head	1.64 (0.82)	16
TAIWAN	Sand	SPT-N = 15	Group (2x3)	Fixed Head	1.47 (0.74)	15
MERSİN	Gravelly Sand	N/A	Single Pile	Free Head	0.63	N/A
ÇATALAĞZI	Alluvium	N/A	Single Pile	Free Head	0.79	N/A
GARSTON	Sandy Gravel	SPT-N = 50 ϕ = 40°	Single Pile	Free Head	0.59	50
TAIWAN	Sand	SPT-N = 15	Single Pile	Free Head	0.65	15
N/A	Sand	N/A	Single Pile	Free Head	0.74	N/A

From the available information provided in Table 3.4, recommendations for sands and clays with different SPT-N values are made as follows:

For clays;

$N < 10 \qquad \beta = 1.00 \sim 0.90$

$10 < N < 20 \qquad \beta = 0.90 \sim 0.80$

For sands;

$10 < N < 20 \qquad \beta = 0.80 \sim 0.70$

$20 < N < 50 \qquad \beta = 0.70 \sim 0.55$

Because of the fact that the number of available data is scarce for such simplified analysis, the recommended β values correspond to relatively higher range of SPT-N values, especially for sands. On the other hand, they are compatible with each other when it is considered that the lateral deflections would decrease with increasing soil resistance. A more definitive recommendation including smaller ranges of SPT-N values can be done by performing additional lateral load tests on piles in soils with different characteristics.

The final step is the application of three factors (α, β, η) to Equations 3.4 and 3.5. These two equations result in the lateral deflection value at the top of the pile. Reliability of the outcome will depend on the soil type and group efficiency factors (Figure 3.3).

3.2.2.4 Checking the suitability of the proposed simple method

A sample concrete pile with a diameter of 1.0 m and modulus of elasticity 25 GPa is assumed to check the suitability of the proposed method. Soil profile is thought to be dense sand with average SPT-N values of 40, for which the soil type factor β can be taken as 0.60. Pile is assumed to be a single pile that no group reduction is applied ($\eta = 1.0$). In addition to these, the loading value is also required to perform an analysis. A loading range between –3000 kN and 3000 kN is selected to plot the p-y curve. For every 1 kN increase in the load, a corresponding deflection is calculated by using Equation 4. When those load versus deflection values are plotted against each other in Figure 3.4, it can be seen that the general trend of p-y curve represents a nonlinear behavior.

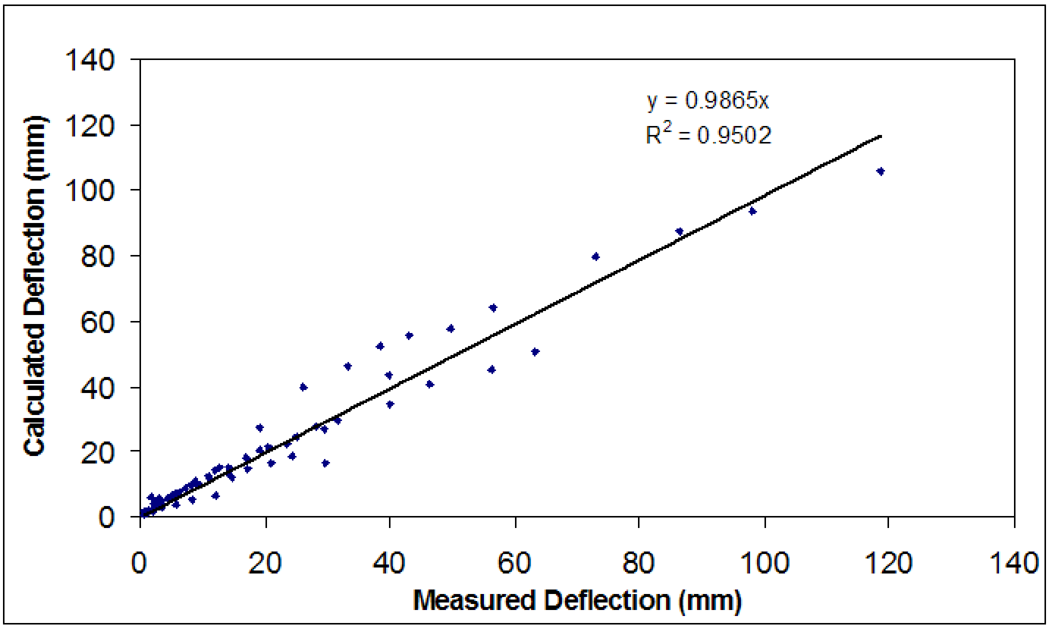


Figure 3.3. Suitability of simple method

Effect of the soil and group reduction can also be checked on the sample model. In order to simulate how the proposed method is affected from changes in soil type, the sample pile is assumed to be in soft clay with SPT-N value of 10. The only change in the formulation is the β factor, which is taken as 0.90 for soft clay. When the case of soft clay is plotted against the case of stiff sand as in Figure 3.5, it can be observed that for the same load, deflection of the pile in soft clay is more than the deflection of the pile in sand. Such a plot is expected since lateral resistance of the stiff sand is higher than the resistance provided by soft clay. Similarly, the group effect is also checked by changing the η value in the original model. A group of piles with a center–to-center spacing of 3 diameters is assumed and a group efficiency factor of 0.5 is selected. As it can be seen in Figure 3.6, the pile group deflects more than the single pile under the same lateral load due to the shadowing effect.

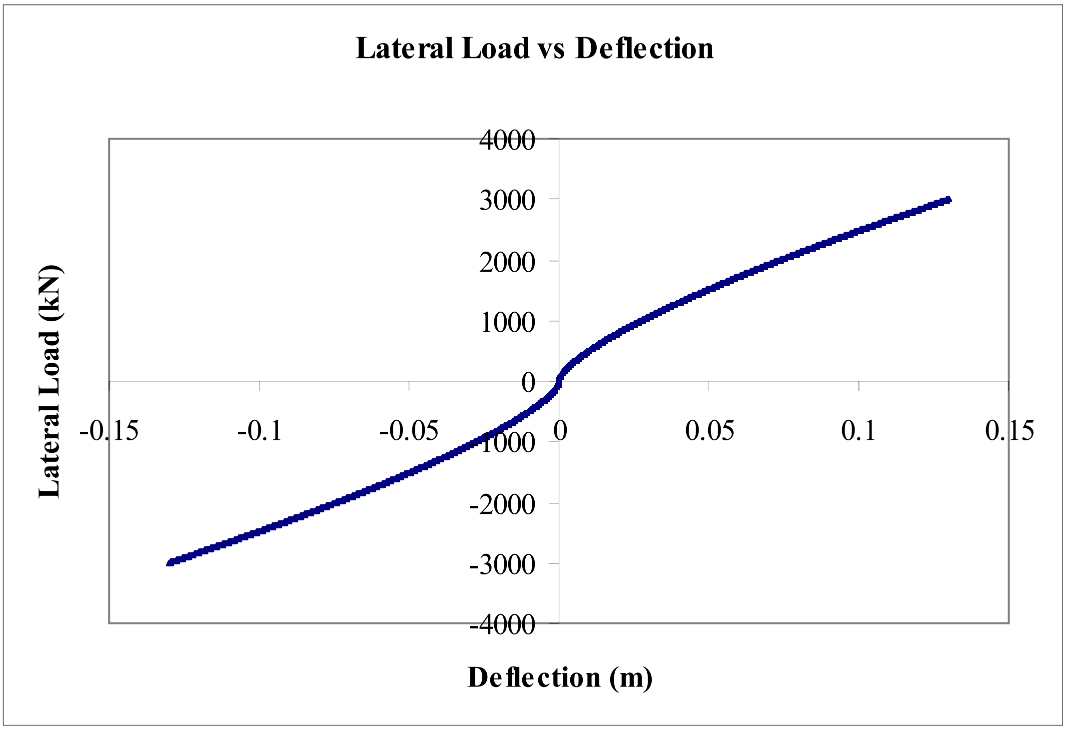


Figure 3.4. p-y curve computed by proposed method

When a structural analysis software is utilized for computation, the common practice is modeling of the pile to its full length and assigning p-y curves at every 1m distance along the pile. The proposed simple method does not require the modeling of the pile system and in the software model. The whole foundation system is represented by a lumped linear spring. The most critical point is the selection of the spring constant for the lumped spring representing the foundation of the structure.

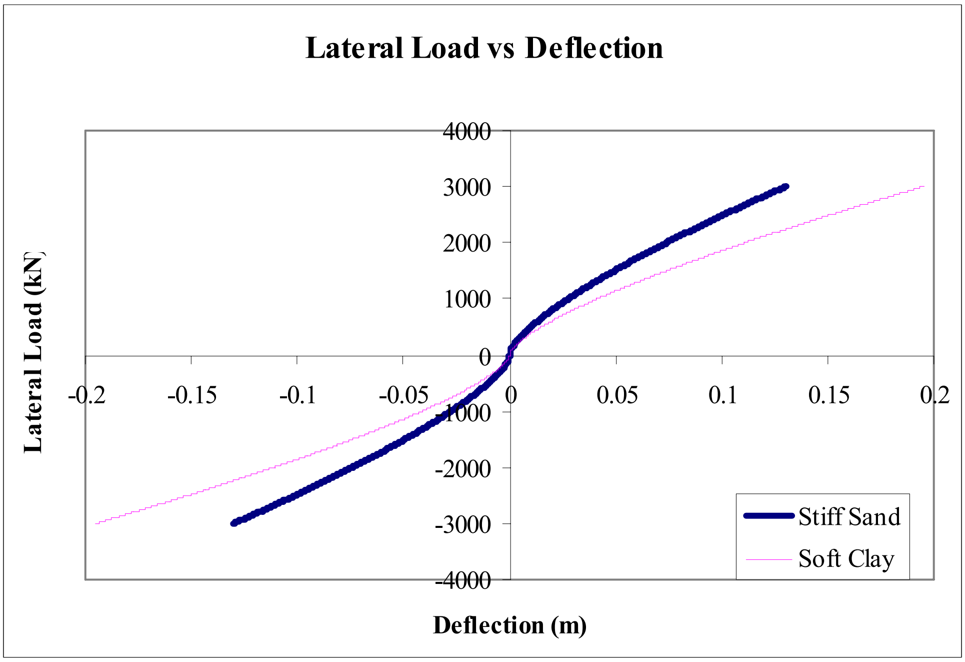


Figure 3.5. Illustration of the effect of soil type (β factor) for the proposed method

As the approximate deflection at the top of the pile or pile system is computed according to the procedure described above and the lateral load from the superstructure is known, the spring constant can be calculated simply by:

$$k = \frac{P}{\delta} \qquad \text{where} \begin{cases} \text{k: spring constant (kN/m)} \\ \delta\text{: lateral deflection at top of the pile (m)} \\ \text{P: lateral load at top of the pile (kN)} \end{cases}$$

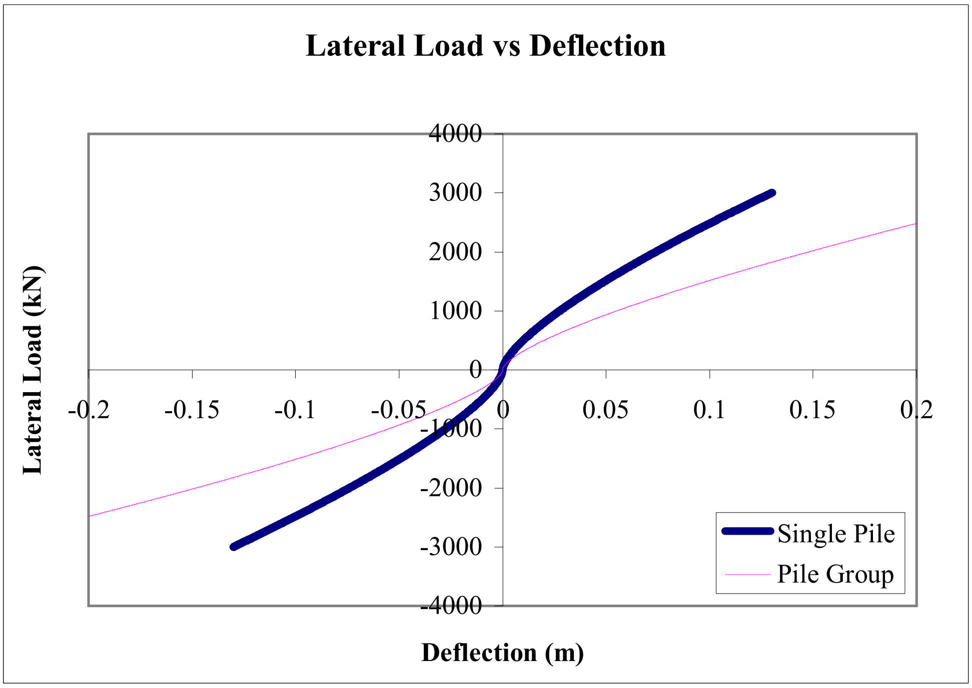


Figure 3.6. Illustration of the effect of group reduction (η factor) for the proposed method

CHAPTER 4

COMPUTER MODELS AND ANALYSIS

4.1 Modelling Methods in Practice

Lateral pile response is typically assessed using commercially available softwares and alternatively, general-purpose structural analysis programs. Soil resistance surrounding piles are simulated by linear, nonlinear or hysteretic springs depending on the analysis method. A more closely spaced soil spring system along the length of a pile may define lateral behavior better than widely spaced soil springs. In some cases, 6x6 soil springs with off-diagonal terms can be used to simulate soil-structure interaction.

Critical lateral load on piles are typically developed due to earthquake rather than service loads. Commonly two types of analyses are performed for determination of lateral response. One of them is response spectrum analysis, which is a linear type, and can be constructed when the provided data for soil conditions and maximum ground acceleration are available. The second type is time history analysis, which can be nonlinear type, requiring the full record of an earthquake. Designers usually perform response spectrum analysis for seismic design of a structure because of time efficiency.

Soil springs used in a response spectrum analysis (RSA) cannot be defined as nonlinear since RSA is an elastic analysis. Therefore, an effective stiffness is used for a given load and deflection as illustrated in Figure 4.1. In the figure, it can be seen that for the initial portion of the load – deflection curve, spring constant may be assigned since this part can be said to be linear. However as load increases, the point at which the effective stiffness is calculated moves to the right of the curve where nonlinear behavior is most effective. On the other hand, time history analysis can use whole nonlinear load – deflection curve.

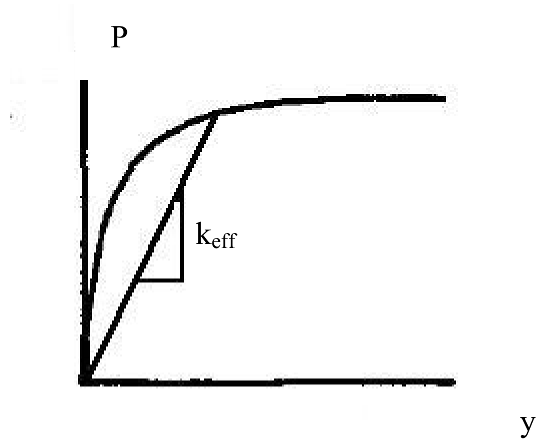


Figure 4.1. Spring stiffness for response spectrum analysis

4.2 Sample Computer Model

A series of bridges were analyzed using LARSA, a general-purpose structural analysis software. The same superstructure was used in all models whereas the substructures varied. The 64 m. long bridge had two spans with two abutments and a 1.5 m diameter single circular pier carrying prestressed beams and slab as shown in Figure 3.2. Four 1.0 m diameter piles spaced at 3 meters were selected for foundation. Pile cap was placed 1.0 m below the ground line considering possible scour of the soil above the pile cap and to keep the foundation away from the frost line.

Abutments are effective on total lateral response when the lateral thrust of the backfill is considered. For a simplified response spectrum analysis, abutment stiffness can be simplified by replacing rigidity of elastomeric pads used in longitudinal direction and effective rigidity of transverse shear key used in

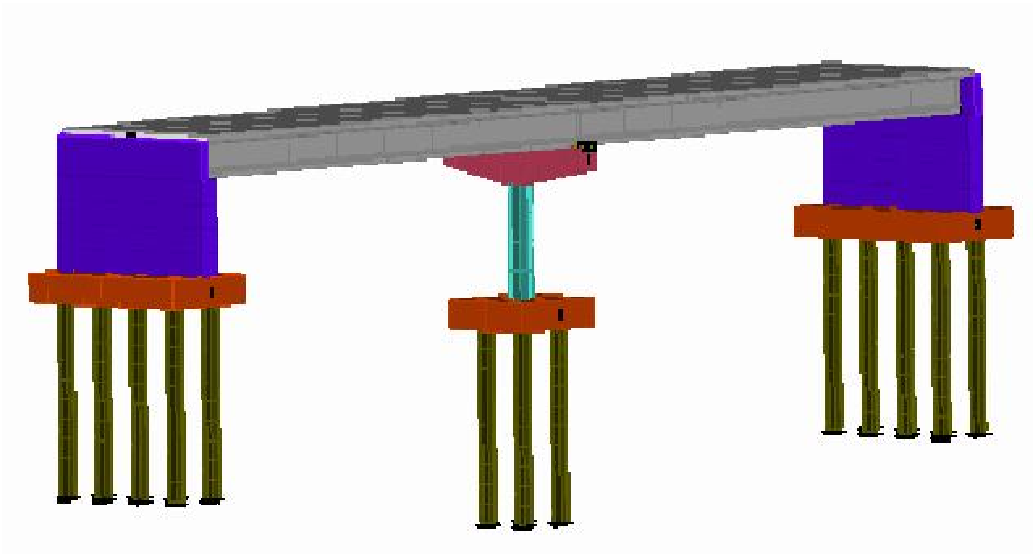


Figure 4.2. A sample bridge model with abutments

transverse direction with abutment as shown in Figure 4.3. The section and side view of the pier were presented in Figures 4.4 and 4.5.

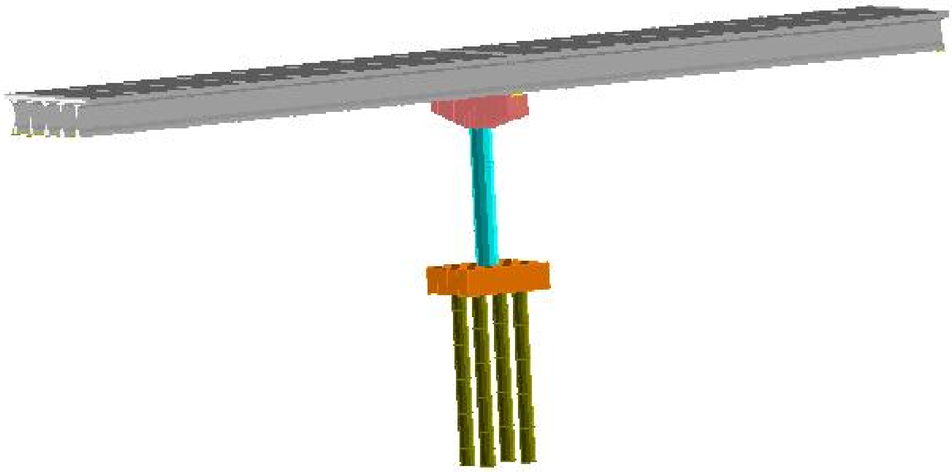


Figure 4.3. Bridge model with simplified abutment model

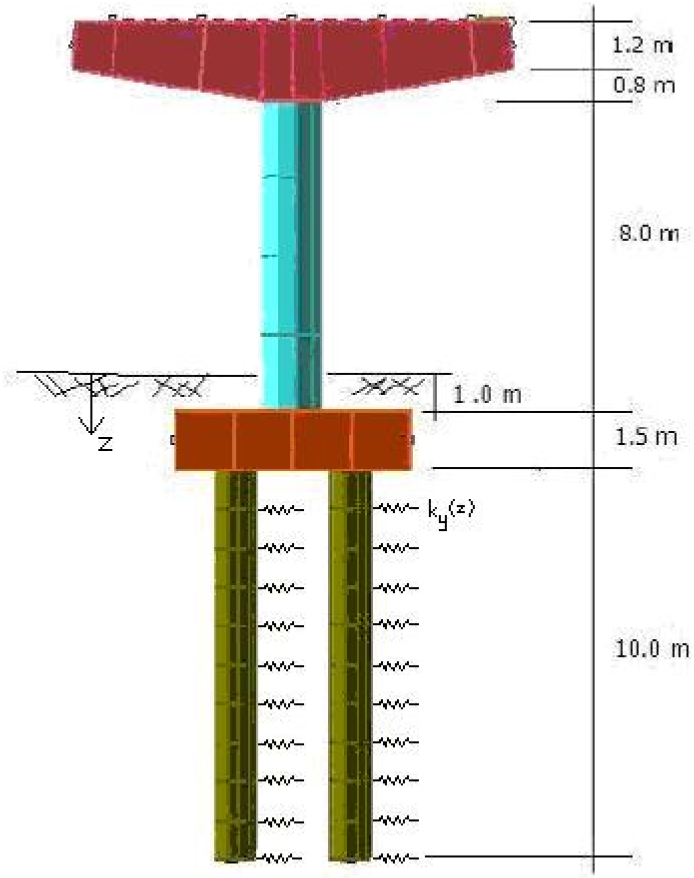


Figure 4.4. Pier elevation

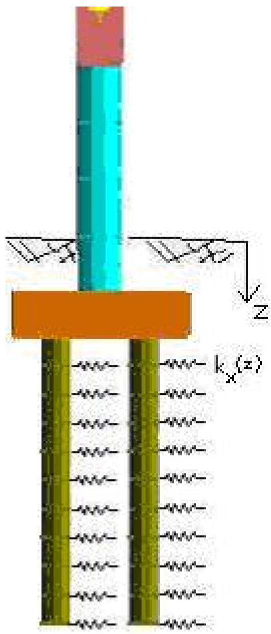


Figure 4.5. Pier side view

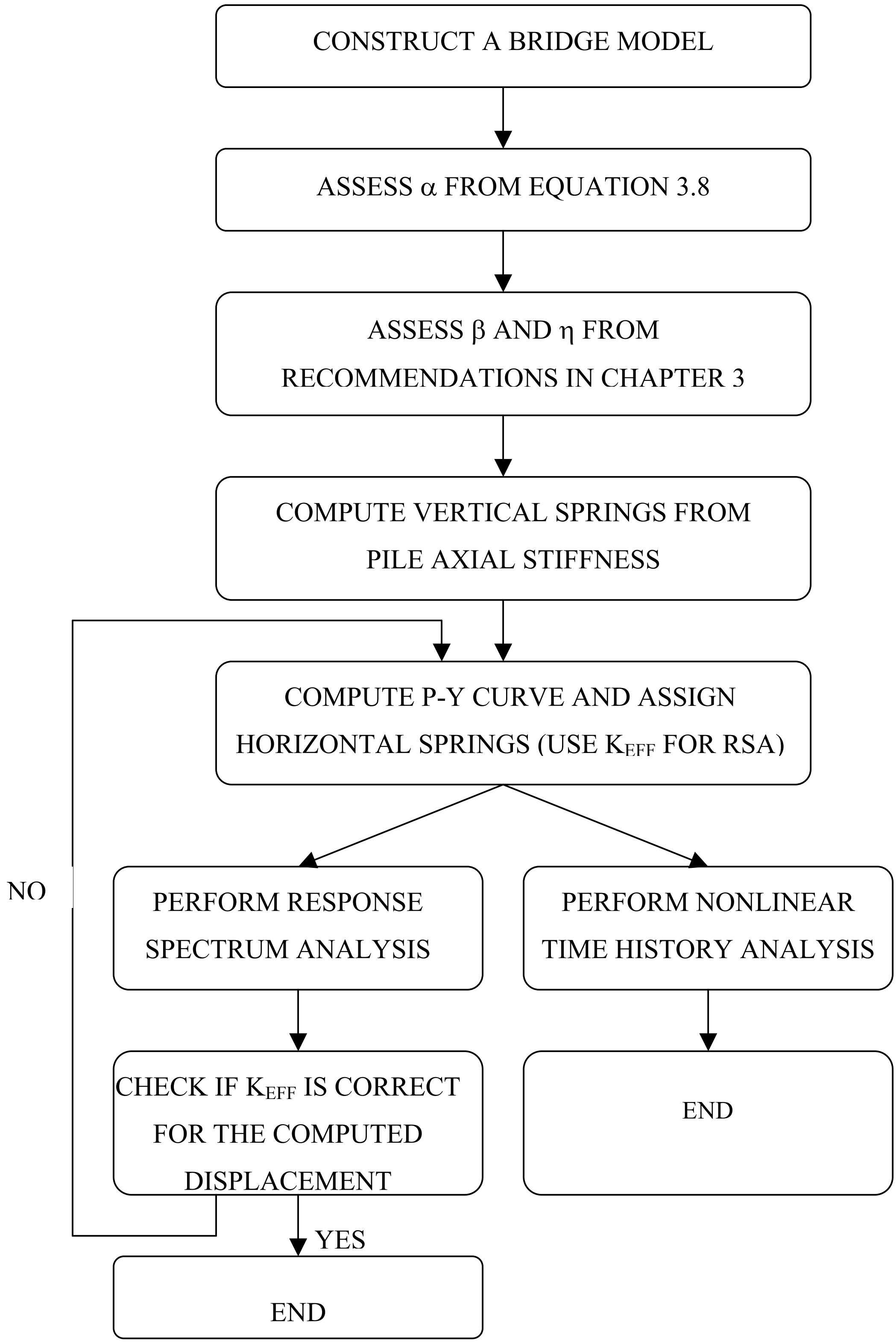


Figure 4.6. Algorithm for analysis

4.3 Response Spectrum Analysis

Foundations of structures at competent soils (N≥20 for granular soils and c_u>72 kPa for cohesive soils) can experience small deformations per Caltrans Seismic Design Criteria. Therefore, soil-structure interaction (SSI) can be ignored for structures at competent soils. However, SSI may be very effective for the case with poor soils (N<10) and shall be included in analysis. Apart from these two soil types, there are also marginal soils, which cannot be classified as competent or poor. For this type of soil, interaction between soil and substructure is needed to be well defined.

Pier stiffness is also an important parameter that may dominantly control the structural response. Flexible pier behaves more like a seismic isolation system and most of the energy is dissipated by deformations that take place in this region. Therefore, the effect of soil-structure interaction will be less effective on the whole system. On the other hand, for the case with rigid columns, most of the deformations will take place at the foundation level.

In the following pages, methods defining the lateral performance of piles in the literature were compared to the results of proposed simple method. Competent and poor soils consisting of sand or clay type of soil profiles were analyzed to define structural response for flexible and rigid column pier cases. Response spectrum analysis was performed using an iterative procedure to define the effective soil spring constants along the pile. Iterations are terminated when selected effective soil spring converges to the target load and deflection. Iterative procedure was demonstrated in detail for competent soil and flexible column case. Procedure and computation steps were the same for all other cases (competent soil with rigid column, poor soil with flexible column, poor soil with rigid column), and the results were summarized at the end. When soil-structure interaction was ignored, a fully fixed support was placed at foundation level, a typical practice in bridge engineering. Results of the analysis with SSI were compared to the results of a fixed foundation to identify impact of SSI on structural response. Pile cap soil resistance was ignored in all analysis.

AASHTO response spectrum was generated for a site where acceleration coefficient (A) is 0.4 and site coefficient (S) is 1.5. Elastic seismic response coefficient (C_s) was plotted against period and response spectrum curve was given in Figure 4.7. Load combinations were introduced to define the response of bridge in both longitudinal and transverse directions.

For longitudinal direction : (DEAD LOAD) + ($EQ_{LONG.}$) + ($0.3EQ_{TRANS.}$)
For transverse direction : (DEAD LOAD) + ($0.3EQ_{LONG.}$) + ($EQ_{TRANS.}$)

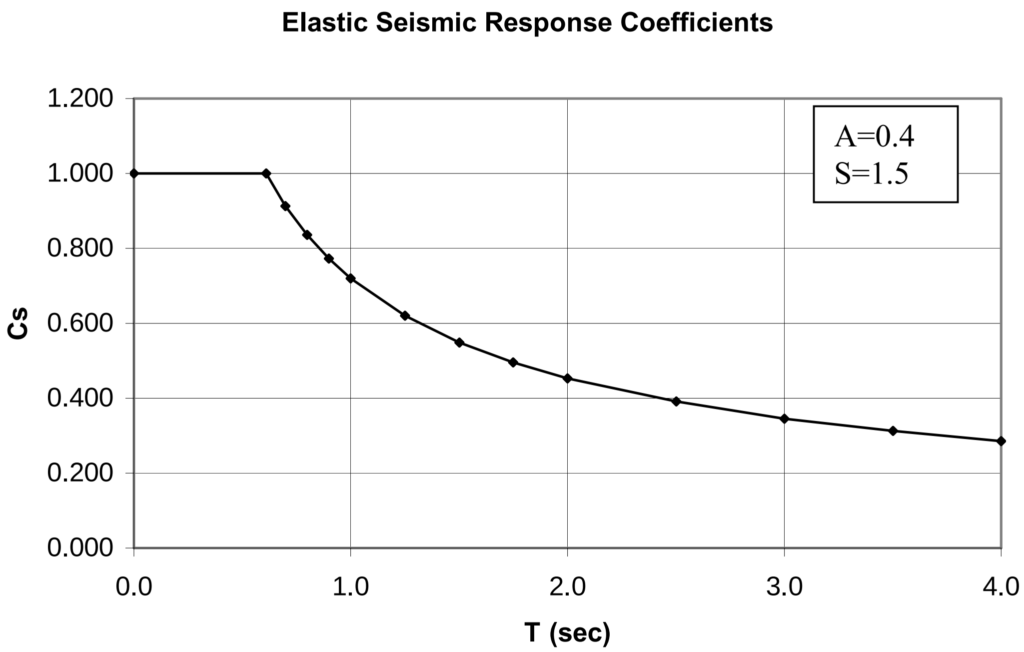


Figure 4.7. AASHTO response spectrum curve

4.3.1 Response Spectrum Analysis at Competent Soil with Flexible Column

4.3.1.1 Sandy Soil

Methods proposed by Reese et al. (1974) and API (1993) were typically used to determine the lateral response of piles in sands. In these methods, nonlinear soil springs throughout the depth of piles were computed for an advance analyses.

Alternatively, a lumped soil spring at the foundation pier intersection could be calculated based on proposed method for a simple analysis.

Soil and pile properties were as follows:

SPT-N = 20

$\phi = 30°$

$\gamma = 20$ kN/m^3

Pile Diameter = 1 m

Pile Length = 10 m. (from the base of pile cap)

Modulus of Elasticity of Pile = 24800000 kN/m^2

Pier Section = 1.5 m (circular) (Flexible Pier)

4.3.1.1.1 Method of Reese et al. (1974)

P-y curves were computed at every 1.0 m distance based on industry practice as demonstrated in Table 4.1. Computation of p-y curves started from 3.0 m depth since the pile cap was embedded. Lateral resistance of the soil was assumed to be constant below 10 m from ground level and p-y curve at 10 m was assigned to the springs at lower elevations. To start iterations, an initial value for soil spring constant "k" was assigned. Using the deflection values at every 1 m, "k" values for next iteration were calculated from corresponding lateral force at p-y curves. This procedure continued until the calculated "k" value was very close to the previous one. P-y curves for the first iteration step were given in Table 4.2. Details of iterations and convergence were summarized in Table 4.3 and Table 4.4.

In case of static loading, shadowing effect takes place, and group efficiency shall be considered in analysis. However, no group reduction was applied for earthquake load combinations regarding the recommendation of ATC32 and Caltrans Bridge Seismic Design Criteria. Similar recommendations were also made in AASHTO-LRFD.

Table 4.1. Summary of parameters used in p-y curve for sand (Reese et al., 1974)

Depth (m)	Φ' (°)	γ' (kN/m^3)	k (kN/m)	P_{st} (kN)	P_{sd} (kN)	P_s (kN)	z/D	A_s	B_s	α	β	K_a	K_0	p_u (kN)	p_m (kN)	y_u (m)	y_m (m)	m	n	y_k (m)
3	30	20	20000	504	1725	504	3	1	0.8	0.26	1.05	0.3	0	524	378	0.04	0.02	7017	0.309	5.00E-05
4	30	20	20000	825	2300	825	4	0.9	0.5	0.26	1.05	0.3	0	734	429	0.04	0.02	14653	0.569	0.00017
5	30	20	20000	1223	2875	1223	5	0.9	0.5	0.26	1.05	0.3	0	1076	611	0.04	0.02	22299	0.608	0.00013
6	30	20	20000	1696	3449	1696	6	0.9	0.5	0.26	1.05	0.3	0	1493	848	0.04	0.02	30943	0.608	7.43E-05
7	30	20	20000	2247	4024	2247	7	0.9	0.5	0.26	1.05	0.3	0	1977	1123	0.04	0.02	40982	0.608	4.61E-05
8	30	20	20000	2874	4599	2874	8	0.9	0.5	0.26	1.05	0.3	0	2529	1437	0.04	0.02	52415	0.608	3.03E-05
9	30	20	20000	3577	5174	3577	9	0.9	0.5	0.26	1.05	0.3	0	3148	1788	0.04	0.02	65244	0.608	2.08E-05
10	30	20	20000	4357	5749	4357	10	0.9	0.5	0.26	1.05	0.3	0	3834	2178	0.04	0.02	79467	0.608	1.48E-05
11	30	20	20000	4357	5749	4357	10	0.9	0.5	0.26	1.05	0.3	0	3834	2178	0.04	0.02	79467	0.608	1.48E-05
12	30	20	20000	4357	5749	4357	10	0.9	0.5	0.26	1.05	0.3	0	3834	2178	0.04	0.02	79467	0.608	1.48E-05

Table 4.2. P-y curves at each 1 m depth along the pile computed by method of Reese et al. (1974)

Depth : 3m		Depth : 4m		Depth : 5m		Depth : 6m		Depth : 7m		Depth : 8m		Depth : 9m		Depth : 10m		Depth : 11m	
P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)
-524	-0.150	-734	-0.150	-1076	-0.150	-1493	-0.150	-1977	-0.150	-2529	-0.150	-3148	-0.150	-3834	-0.150	-3834	-0.150
-524	-0.038	-734	-0.038	-1076	-0.038	-1493	-0.038	-1977	-0.038	-2529	-0.038	-3148	-0.038	-3834	-0.038	-3834	-0.038
-495	-0.033	-673	-0.033	-983	-0.033	-1364	-0.033	-1806	-0.033	-2310	-0.033	-2876	-0.033	-3503	-0.033	-3503	-0.033
-466	-0.029	-612	-0.029	-890	-0.029	-1235	-0.029	-1636	-0.029	-2092	-0.029	-2604	-0.029	-3172	-0.029	-3172	-0.029
-437	-0.025	-551	-0.025	-797	-0.025	-1106	-0.025	-1465	-0.025	-1874	-0.025	-2332	-0.025	-2841	-0.025	-2841	-0.025
-378	-0.017	-490	-0.021	-704	-0.021	-977	-0.021	-1294	-0.021	-1655	-0.021	-2060	-0.021	-2509	-0.021	-2509	-0.021
-370	-0.016	-382	-0.014	-540	-0.014	-753	-0.014	-1004	-0.014	-1294	-0.014	-1624	-0.014	-2178	-0.017	-2178	-0.017
-362	-0.014	-329	-0.010	-462	-0.011	-650	-0.011	-874	-0.011	-1139	-0.011	-1448	-0.012	-1807	-0.012	-1807	-0.012
-353	-0.013	-270	-0.007	-374	-0.007	-534	-0.008	-731	-0.008	-970	-0.009	-1258	-0.009	-1601	-0.010	-1601	-0.010
-344	-0.012	-198	-0.004	-271	-0.004	-399	-0.005	-566	-0.005	-779	-0.006	-1046	-0.007	-1377	-0.008	-1377	-0.008
-270	-0.006	-180	-0.004	-173	-0.002	-157	-0.001	-116	0.000	-63	0.000	-30	0.000	-1126	-0.006	-1126	-0.006
270	0.006	180	0.004	173	0.002	157	0.001	116	0.000	63	0.000	30	0.000	1126	0.006	1126	0.006
344	0.012	198	0.004	271	0.004	399	0.005	566	0.005	779	0.006	1046	0.007	1377	0.008	1377	0.008
353	0.013	270	0.007	374	0.007	534	0.008	731	0.008	970	0.009	1258	0.009	1601	0.010	1601	0.010
362	0.014	329	0.010	462	0.011	650	0.011	874	0.011	1139	0.011	1448	0.012	1807	0.012	1807	0.012
370	0.016	382	0.014	540	0.014	753	0.014	1004	0.014	1294	0.014	1624	0.014	2178	0.017	2178	0.017
378	0.017	490	0.021	704	0.021	977	0.021	1294	0.021	1655	0.021	2060	0.021	2509	0.021	2509	0.021
437	0.025	551	0.025	797	0.025	1106	0.025	1465	0.025	1874	0.025	2332	0.025	2841	0.025	2841	0.025
466	0.029	612	0.029	890	0.029	1235	0.029	1636	0.029	2092	0.029	2604	0.029	3172	0.029	3172	0.029
495	0.033	673	0.033	983	0.033	1364	0.033	1806	0.033	2310	0.033	2876	0.033	3503	0.033	3503	0.033
524	0.038	734	0.038	1076	0.038	1493	0.038	1977	0.038	2529	0.038	3148	0.038	3834	0.038	3834	0.038
524	0.150	734	0.150	1076	0.150	1493	0.150	1977	0.150	2529	0.150	3148	0.150	3834	0.150	3834	0.150

Table 4.3. “k” values found by iteration for Reese et al. method

		3 m	4 m	5 m	6 m	7 m	8 m	9 m	10 m	11 m
ITERATION 1	Long	60000	80000	100000	120000	140000	160000	180000	200000	200000
	Trans	60000	80000	100000	120000	140000	160000	180000	200000	200000
ITERATION 2	Long	63699	58726	97062	182219	368722	160000	180000	200000	200000
	Trans	32701	38566	66090	124774	272518	160000	180000	200000	200000
ITERATION 3	Long	62828	58161	96842	187492	441789	160000	180000	200000	200000
	Trans	26124	32617	55151	100556	203515	160000	180000	200000	200000
ITERATION 4	Long	62647	58066	96806	188277	456456	160000	180000	200000	200000
	Trans	24185	30771	51560	91662	174394	160000	180000	200000	200000
ITERATION 5	Long	62608	58045	96791	188392	459157	160000	180000	200000	200000
	Trans	23505	30111	50268	88481	164677	160000	180000	200000	200000
ITERATION 6	Long	62599	58039	96785	188403	459606	160000	180000	200000	200000
	Trans	23254	29868	49796	87336	161329	160000	180000	200000	200000
ITERATION 7	Long	62596	58037	96782	188401	459662	160000	180000	200000	200000
	Trans	23162	29778	49622	86920	160135	160000	180000	200000	200000
ITERATION 8	Long	62595	58037	96781	188398	459659	160000	180000	200000	200000
	Trans	23128	29745	49558	86767	159701	160000	180000	200000	200000

Table 4.4. Design parameters from iterative response spectrum analysis for method of Reese et al. (1974)

	M_{long} (kN.m)	M_{trans} (kN.m)	δ_{long} (m)	δ_{trans} (m)
ITERATION 1	15349	23964	0.006	0.004
ITERATION 2	15345	23807	0.006	0.006
ITERATION 3	15344	23736	0.006	0.006
ITERATION 4	15344	23709	0.006	0.006
ITERATION 5	15344	23699	0.006	0.006
ITERATION 6	15344	23695	0.006	0.007
ITERATION 7	15344	23694	0.006	0.007
ITERATION 8	15344	23693	0.006	0.007

4.3.1.1.3 Method of American Petroleum Institute (1993)

Another commonly used method for determination of lateral response of piles in sands was suggested by API. Calculation effort in this method was less than the method suggested by Reese et al. (1974). In API (1993) design aid charts were provided based on empirical data, a function of internal angle of friction. The detailed procedure was described in Appendix A.2.2. The same response spectrum analysis was performed to assess the structural response.

The computation process was summarized in Table 4.5, which was formed in a spreadsheet, used for easiness in calculations. This method also requires an iterative solution similar to the one used at Reese et al. method. C_1, C_2, C_3 parameters were obtained from Figure A.9 in Appendix A.2.2. Computed p-y curves for the first iteration were presented in Table 4.6.

Table 4.5. Summary of parameters to be used in p-y curve for sand. (API, 1993)

Depth (m)	γ' (kN/m^3)	ϕ' (°)	k (kN/m)	C_1	C_2	C_3	P_u (kN)	A
3	20	30	20000	1.857	2.71	28.571	497	0.9
4	20	30	20000	1.857	2.71	28.571	811	0.9
5	20	30	20000	1.857	2.71	28.571	1200	0.9
6	20	30	20000	1.857	2.71	28.571	1662	0.9
7	20	30	20000	1.857	2.71	28.571	2199	0.9
8	20	30	20000	1.857	2.71	28.571	2811	0.9
9	20	30	20000	1.857	2.71	28.571	3496	0.9
10	20	30	20000	1.857	2.71	28.571	4256	0.9
11	20	30	20000	1.857	2.71	28.571	4256	0.9
12	20	30	20000	1.857	2.71	28.571	4256	0.9

As summarized in Table 4.7, method of API has a high convergence rate. Only 4 iterations were enough for the computation of total response. Results (Table 4.8) were comparable to the ones computed for method of Reese et al. API method seemed to be more time efficient when calculation effort and convergence rate was considered.

Table 4.6. P-y curves at each 1 m depth throughout the pile computed by method of API (1993)

Depth : 3m		Depth : 4m		Depth : 5m		Depth : 6m		Depth : 7m		Depth : 8m		Depth : 9m		Depth : 10m	
P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)
-447	-0.1	-730	-0.100	-1080	-0.100	-1496	-0.100	-1979	-0.100	-2529	-0.100	-3146	-0.100	-3830	-0.100
-447	-0.04	-730	-0.040	-1078	-0.040	-1491	-0.040	-1966	-0.040	-2498	-0.040	-3082	-0.040	-3715	-0.040
-443	-0.02	-712	-0.020	-1028	-0.020	-1380	-0.020	-1759	-0.020	-2156	-0.020	-2567	-0.020	-2986	-0.020
-435	-0.02	-687	-0.016	-974	-0.016	-1283	-0.016	-1606	-0.016	-1939	-0.016	-2277	-0.016	-2618	-0.016
-413	-0.01	-632	-0.012	-869	-0.012	-1115	-0.012	-1367	-0.012	-1620	-0.012	-1874	-0.012	-2128	-0.012
-390	-0.01	-583	-0.010	-787	-0.010	-995	-0.010	-1205	-0.010	-1416	-0.010	-1626	-0.010	-1836	-0.010
-354	-0.01	-514	-0.008	-680	-0.008	-847	-0.008	-1014	-0.008	-1181	-0.008	-1347	-0.008	-1513	-0.008
-298	-0.01	-421	-0.006	-545	-0.006	-669	-0.006	-793	-0.006	-916	-0.006	-1039	-0.006	-1162	-0.006
-219	-0	-301	-0.004	-383	-0.004	-464	-0.004	-546	-0.004	-627	-0.004	-708	-0.004	-789	-0.004
-117	-0	-157	-0.002	-198	-0.002	-238	-0.002	-278	-0.002	-318	-0.002	-358	-0.002	-399	-0.002
0	0	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000
117	0.002	157	0.002	198	0.002	238	0.002	278	0.002	318	0.002	358	0.002	399	0.002
219	0.004	301	0.004	383	0.004	464	0.004	546	0.004	627	0.004	708	0.004	789	0.004
298	0.006	421	0.006	545	0.006	669	0.006	793	0.006	916	0.006	1039	0.006	1162	0.006
354	0.008	514	0.008	680	0.008	847	0.008	1014	0.008	1181	0.008	1347	0.008	1513	0.008
390	0.01	583	0.010	787	0.010	995	0.010	1205	0.010	1416	0.010	1626	0.010	1836	0.010
413	0.012	632	0.012	869	0.012	1115	0.012	1367	0.012	1620	0.012	1874	0.012	2128	0.012
435	0.016	687	0.016	974	0.016	1283	0.016	1606	0.016	1939	0.016	2277	0.016	2618	0.016
443	0.02	712	0.020	1028	0.020	1380	0.020	1759	0.020	2156	0.020	2567	0.020	2986	0.020
447	0.04	730	0.040	1078	0.040	1491	0.040	1966	0.040	2498	0.040	3082	0.040	3715	0.040
447	0.1	730	0.100	1080	0.100	1496	0.100	1979	0.100	2529	0.100	3146	0.100	3830	0.100

Table 4.7. "k" values found by iteration for API method

		3 m	4 m	5 m	6 m	7 m	8 m	9 m	10 m	11 m
ITERATION 1	**Long**	60000	80000	100000	120000	140000	160000	180000	200000	200000
	Trans	60000	80000	100000	120000	140000	160000	180000	200000	200000
ITERATION 2	**Long**	55429	78121	99450	119893	139989	159997	179997	199999	199999
	Trans	39456	68686	96240	119269	139948	159995	179984	199992	199992
ITERATION 3	**Long**	55175	77987	99402	119882	139988	159997	179997	199999	199999
	Trans	35782	65071	94489	118800	139887	159998	179984	199990	199990
ITERATION 4	**Long**	55160	77979	99399	119881	139987	159997	179997	199999	199999
	Trans	34879	64077	93955	118645	139864	159999	179984	199990	199990
ITERATION 5	**Long**	55159	77978	99399	119881	139987	159997	179997	199999	199999
	Trans	34639	63804	93804	118600	139857	159999	179984	199990	199990
ITERATION 6	**Long**	55159	77978	99399	119881	139987	159997	179997	199999	199999
	Trans	34574	63730	93763	118588	139856	159999	179984	199990	199990

Table 4.8. Design parameters from iterative response spectrum analysis in sand for method of API

	M_{long} (kN.m)	M_{trans} (kN.m)	δ_{long} (m)	δ_{trans} (m)
ITERATION 1	15349	23964	0.006	0.004
ITERATION 2	15344	23904	0.006	0.005
ITERATION 3	15344	23888	0.006	0.005
ITERATION 4	15344	23884	0.006	0.005
ITERATION 5	15344	23883	0.006	0.005
ITERATION 6	15344	23882	0.006	0.005

4.3.1.1.3 Proposed Simple Method

This method could be considered to be more advantageous in identification of structural response compared to the other methods in terms of time efficiency. Lumped springs in two orthogonal directions were used at foundation level to simulate an equivalent response that could be determined from an advanced pile spring model (Figure 4.8). There was no need for computation of p-y curves at every 1 m depth. Moreover, modelling of piles was not required in the software. The analysis included an iterative solution similar to the two other methods. At each step the lateral load at the foundation level joint was recorded, and deflection was calculated per Equation 3.5. In this computation, α factor was calculated from Equation 3.8 and β factor was taken as 0.70 for the investigated sandy soil type. In previous chapter, a group reduction factor of 0.50 was recommended for pile groups where piles were spaced at 3 diameters but for seismic loading no group reduction was needed and η factor was set to be equal to 1.0. Effective moment of inertia of piles was assumed to be half of the gross moment of inertia. The first step of the iterative procedure was summarized below. Following steps were the same as the first one and the resulting spring constants were summarized in Table 4.9.

<u>1st iteration step</u>:

Initially, the spring constant at the ground level was assumed to be 20000 kN/m in both longitudinal and transverse directions. From analysis results:

$V_x = 520$ kN

$V_y = 1464$ kN

Using Equation 3.8;

$\alpha_x = 0.65*\ln(520)-0.10 = 3.965$

$\alpha_y = 0.65*\ln(1464)-0.10 = 4.638$

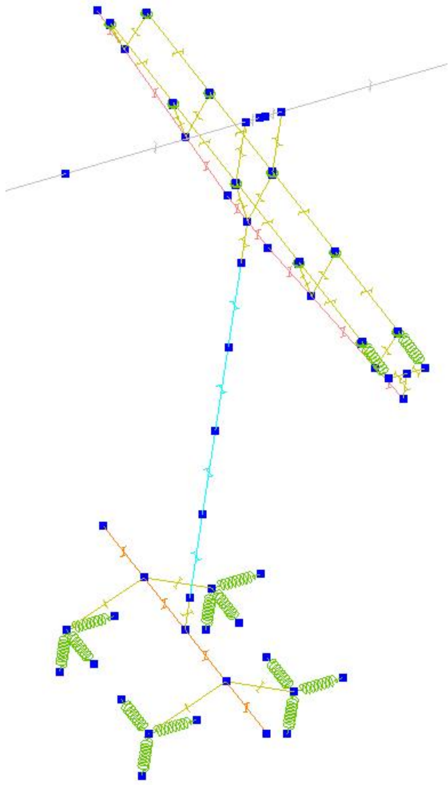


Figure 4.8. Modelling of simple method

Using Equation 3.5;

$$y_x = \left(\frac{520 \times 3.965^3 \times 1^3}{12 \times 24800000 \times \left(\frac{\pi \times 1^4}{64} \right) \times 0.5} \right) \times \frac{0.70}{1.00}$$

$$y_x = 0.0031m$$

$$y_y = \left(\frac{1464 \times 4.638^3 \times 1^3}{12 \times 24800000 \times \left(\frac{\pi \times 1^4}{64} \right) \times 0.5} \right) \times \frac{0.70}{1.00}$$

$$y_y = 0.014m$$

Spring constants for next iteration step:

$$k_x = \frac{V_x}{y_x} = \frac{520}{0.0031} = 167741 kN/m$$

$$k_y = \frac{V_y}{y_y} = \frac{1464}{0.014} = 104571 kN/m$$

Table 4.9. Spring constants iteration by the proposed method for sand

	LONG. DIRECTION SPRING	TRANS DIRECTION SPRING
ITERATION 1	60000	60000
ITERATION 2	167741	104571
ITERATION 3	167404	104999
ITERATION 4	167403	104976
ITERATION 5	167404	104984

The formulation provided in Chapter 3 was for lateral translation only. However, there also occurs a rotation at the foundation level. To account for this rotation, an additional vertical spring was attached to each pile in the model. The constant of this spring is calculated from vertical deflection of piles as:

$$k_v = \left(\frac{E_p A_p}{L_p} \right) \quad \text{(Equation 4.1)}$$

where;

k_v : Vertical spring constant for the spring in the simple model.

E_p : Modulus of elasticity of pile.

A_p : Cross sectional area of pile.

L_p : Length of pile.

$$k_v = \left(\frac{24800000 \times \left(\pi \times 0.5^2\right)}{10} \right) \cong 1950000 kN/m$$

Table 4.10. Design parameters from iterative response spectrum analysis in sand for simple method

	M_{long} (kN.m)	M_{trans} (kN.m)	δ_{long} (m)	δ_{trans} (m)
ITERATION 1	15309	23976	0.009	0.008
ITERATION 2	15449	24123	0.004	0.005
ITERATION 3	15449	24123	0.004	0.005
ITERATION 4	15449	24123	0.004	0.005
ITERATION 5	15449	24123	0.004	0.005

4.3.1.2 Clayey Soil

Methods proposed by Matlock (1970) and API (1993) were used for the determination of the soil springs. The proposed method was applied on the model to check how accurate the design parameters would be when compared to the methods in the literature. Selected clay for the analysis and the pile foundation had the following properties:

SPT-N = 15

c_u = 90 kPa

γ = 20 kN/m^3

Pile Diameter = 1 m

Pile Length = 10 m. (from the base of pile cap)

Modulus of Elasticity of Pile = 24800000 kN/m^2

Pier Section = 1.5 m (circular) (Flexible Pier)

4.3.1.2.3 Method of Matlock (1970)

This method was typically used for determination of p-y curves in clays. As in the case of sands, there were some soil parameters required for analysis such as the laboratory tested undrained shear strength of clay and the strain value corresponding to one half of the maximum principal stress difference (ε_{50}). Per recommendation of Matlock, this value was taken to be 0.005. An iterative procedure similar to the sandy soil models were used for bridges at clay soils.

The principal difference between p-y curves for clays and sands was that, stiffness of sands were linearly dependent on the depth whereas for clays stiffness values were independent of the depth. For illustration, p-y curve at 3 m depth was shown in Figure 4.9. Curves at all depths were calculated by the help of a simple computer code for time efficiency and the resulting curves were summarized in Table 4.12.

Table 4.11. Summary of parameters for p-y curve in clay (Matlock, 1970)

Depth (m)	c_u (kPa)	γ' (kN/m^3)	ε_{50}	y_{50} (m)	J	z_r (m)	P_u (kN)	$8y_{50}$ (m)
3	90	20	0.005	0.0125	0.5	8.307692	465	0.1
4	90	20	0.005	0.0125	0.5	8.307692	530	0.1
5	90	20	0.005	0.0125	0.5	8.307692	595	0.1
6	90	20	0.005	0.0125	0.5	8.307692	660	0.1
7	90	20	0.005	0.0125	0.5	8.307692	725	0.1
8	90	20	0.005	0.0125	0.5	8.307692	790	0.1
9	90	20	0.005	0.0125	0.5	8.307692	810	0.1
10	90	20	0.005	0.0125	0.5	8.307692	810	0.1
11	90	20	0.005	0.0125	0.5	8.307692	810	0.1
12	90	20	0.005	0.0125	0.5	8.307692	810	0.1

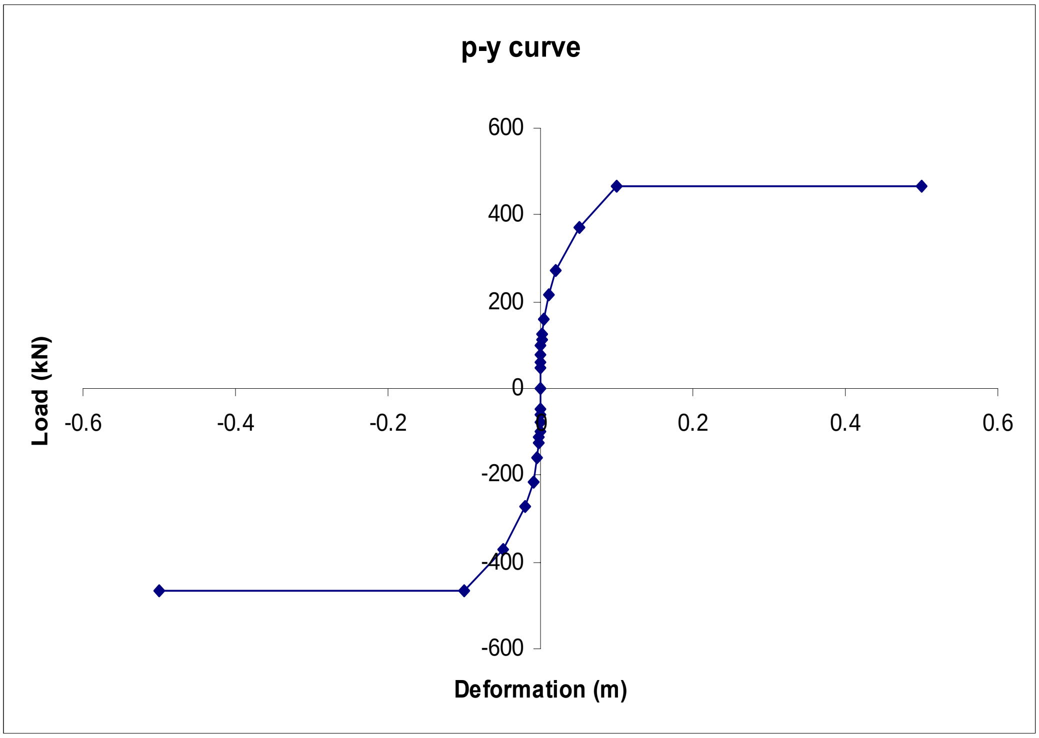


Figure 4.9. P-y curve at 3 m depth (Calculated by the method of Matlock (1970)

Table 4.12. P-y curves at each 1 m depth throughout the pile computed by method of Matlock (1970)

Depth : 3m		Depth : 4m		Depth : 5m		Depth : 6m		Depth : 7m		Depth : 8m		Depth : 9m		Depth : 10m	
P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)
-465	-0.500	-530	-0.500	-595	-0.500	-660	-0.500	-725	-0.500	-790	-0.500	-810	-0.500	-810	-0.500
-465	-0.100	-530	-0.100	-595	-0.100	-660	-0.100	-725	-0.100	-790	-0.100	-810	-0.100	-810	-0.100
-369	-0.050	-421	-0.050	-472	-0.050	-524	-0.050	-575	-0.050	-627	-0.050	-643	-0.050	-643	-0.050
-272	-0.020	-310	-0.020	-348	-0.020	-386	-0.020	-424	-0.020	-462	-0.020	-474	-0.020	-474	-0.020
-216	-0.010	-246	-0.010	-276	-0.010	-306	-0.010	-337	-0.010	-367	-0.010	-376	-0.010	-376	-0.010
-159	-0.004	-181	-0.004	-203	-0.004	-226	-0.004	-248	-0.004	-270	-0.004	-277	-0.004	-277	-0.004
-126	-0.002	-144	-0.002	-162	-0.002	-179	-0.002	-197	-0.002	-214	-0.002	-220	-0.002	-220	-0.002
-100	-0.001	-126	-0.001	-141	-0.001	-157	-0.001	-172	-0.001	-187	-0.001	-192	-0.001	-192	-0.001
-80	-0.001	-91	-0.001	-102	-0.001	-113	-0.001	-124	-0.001	-135	-0.001	-139	-0.001	-139	-0.001
-59	0.000	-67	0.000	-75	0.000	-83	0.000	-91	0.000	-100	0.000	-102	0.000	-102	0.000
-47	0.000	-53	0.000	-60	0.000	-66	0.000	-73	0.000	-79	0.000	-81	0.000	-81	0.000
47	0.000	53	0.000	60	0.000	66	0.000	73	0.000	79	0.000	81	0.000	81	0.000
59	0.000	67	0.000	75	0.000	83	0.000	91	0.000	100	0.000	102	0.000	102	0.000
80	0.001	91	0.001	102	0.001	113	0.001	124	0.001	135	0.001	139	0.001	139	0.001
100	0.001	126	0.001	141	0.001	157	0.001	172	0.001	187	0.001	192	0.001	192	0.001
126	0.002	144	0.002	162	0.002	179	0.002	197	0.002	214	0.002	220	0.002	220	0.002
159	0.004	181	0.004	203	0.004	226	0.004	248	0.004	270	0.004	277	0.004	277	0.004
216	0.010	246	0.010	276	0.010	306	0.010	337	0.010	367	0.010	376	0.010	376	0.010
272	0.020	310	0.020	348	0.020	386	0.020	424	0.020	462	0.020	474	0.020	474	0.020
369	0.050	421	0.050	472	0.050	524	0.050	575	0.050	627	0.050	643	0.050	643	0.050
465	0.100	530	0.100	595	0.100	660	0.100	725	0.100	790	0.100	810	0.100	810	0.100
465	0.500	530	0.500	595	0.500	660	0.500	725	0.500	790	0.500	810	0.500	810	0.500

Table 4.13. "k" values found by iteration for Matlock (1970) method

		3 m	4 m	5 m	6 m	7 m	8 m	9 m	10 m	11 m
ITERATION 1	Long	20000	20000	20000	20000	20000	20000	20000	20000	20000
	Trans	20000	20000	20000	20000	20000	20000	20000	20000	20000
ITERATION 2	Long	23205	30445	40967	56785	81767	124215	263950	348571	348571
	Trans	12117	15892	21426	29820	43230	66357	105521	193898	193898
ITERATION 3	Long	28579	39476	57474	89645	154508	314744	1941931	1223070	1223070
	Trans	11656	15420	21229	30714	47486	80841	152669	393443	393443
ITERATION 4	Long	31975	45409	68968	115030	222855	591498	7215120	1424816	1424816
	Trans	11656	15446	21344	31107	48777	85401	171827	554790	554790
ITERATION 5	Long	33917	48928	76124	131961	273428	837037	11086762	1949086	1949086
	Trans	11693	15507	21461	31360	49414	87375	180151	655720	655720
ITERATION 6	Long	34992	50929	80357	142640	309240	1059623	13715566	2540349	2540349
	Trans	11721	15551	21538	31514	49770	88400	184310	714592	714592
ITERATION 7	Long	35589	52059	82812	149131	333104	1247808	15475784	3091120	3091120
	Trans	11738	15579	21586	31606	49975	88969	186565	748879	748879
ITERATION 8	Long	35923	52698	84226	152988	348233	1392225	16656503	3551692	3551692
	Trans	11748	15595	21613	31659	50092	89293	187834	768951	768951
ITERATION 9	Long	36112	53061	85039	155253	357504	1493979	17463997	3906880	3906880
	Trans	11754	15604	21629	31689	50160	89478	188557	780691	780691
ITERATION 10	Long	36220	53269	85508	156574	363062	1560966	18035178	4164846	4164846
	Trans	11757	15610	21639	31707	50199	89584	188973	787534	787534

Table 4.14. Design parameters from iterative response spectrum analysis for method of Matlock (1970)

	M_{long} (kN.m)	M_{trans} (kN.m)	δ_{long} (m)	δ_{trans} (m)
ITERATION 1	15205	23450	0.011	0.009
ITERATION 2	15256	23327	0.009	0.009
ITERATION 3	15285	23320	0.008	0.009
ITERATION 4	15299	23323	0.007	0.009
ITERATION 5	15306	23326	0.007	0.009
ITERATION 6	15310	23328	0.007	0.009
ITERATION 7	15312	23330	0.007	0.009
ITERATION 8	15313	23330	0.007	0.009
ITERATION 9	15314	23331	0.007	0.009
ITERATION 10	15314	23331	0.007	0.009

Table 4.15. Summary of parameters to be used in p-y curve for clay (API, 1993)

Depth (m)	c (kPa)	ε_{50}	γ' (kN/m^3)	X_R (m)	p_u (kN)	y_c (m)
3	90	0.005	20	9.642857	438	0.0125
4	90	0.005	20	9.642857	494	0.0125
5	90	0.005	20	9.642857	550	0.0125
6	90	0.005	20	9.642857	606	0.0125
7	90	0.005	20	9.642857	662	0.0125
8	90	0.005	20	9.642857	718	0.0125
9	90	0.005	20	9.642857	774	0.0125
10	90	0.005	20	9.642857	810	0.0125
11	90	0.005	20	9.642857	810	0.0125
12	90	0.005	20	9.642857	810	0.0125

Table 4.16. P-y curves at each 1 m depth throughout the pile computed by method of API (1993)

Depth : 3m		Depth : 4m		Depth : 5m		Depth : 6m		Depth : 7m		Depth : 8m		Depth : 9m		Depth : 10m	
P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)	P (kN)	y (m)
-438	-0.150	-494	-0.150	-550	-0.150	-606	-0.150	-662	-0.150	-718	-0.150	-774	-0.150	-810	-0.150
-438	-0.100	-494	-0.100	-550	-0.100	-606	-0.100	-662	-0.100	-718	-0.100	-774	-0.100	-810	-0.100
-360	-0.060	-405	-0.060	-451	-0.060	-497	-0.060	-543	-0.060	-589	-0.060	-635	-0.060	-665	-0.060
-340	-0.050	-383	-0.050	-427	-0.050	-470	-0.050	-514	-0.050	-557	-0.050	-601	-0.050	-629	-0.050
-320	-0.040	-361	-0.040	-402	-0.040	-443	-0.040	-484	-0.040	-525	-0.040	-566	-0.040	-592	-0.040
-286	-0.030	-323	-0.030	-360	-0.030	-396	-0.030	-433	-0.030	-470	-0.030	-506	-0.030	-530	-0.030
-248	-0.020	-280	-0.020	-311	-0.020	-343	-0.020	-375	-0.020	-406	-0.020	-438	-0.020	-458	-0.020
-175	-0.010	-198	-0.010	-220	-0.010	-242	-0.010	-265	-0.010	-287	-0.010	-310	-0.010	-324	-0.010
-88	-0.005	-99	-0.005	-110	-0.005	-121	-0.005	-132	-0.005	-144	-0.005	-155	-0.005	-162	-0.005
-35	-0.002	-40	-0.002	-44	-0.002	-48	-0.002	-53	-0.002	-57	-0.002	-62	-0.002	-65	-0.002
-18	-0.001	-20	-0.001	-22	-0.001	-24	-0.001	-26	-0.001	-29	-0.001	-31	-0.001	-32	-0.001
0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000
18	0.001	20	0.001	22	0.001	24	0.001	26	0.001	29	0.001	31	0.001	32	0.001
35	0.002	40	0.002	44	0.002	48	0.002	53	0.002	57	0.002	62	0.002	65	0.002
88	0.005	99	0.005	110	0.005	121	0.005	132	0.005	144	0.005	155	0.005	162	0.005
175	0.010	198	0.010	220	0.010	242	0.010	265	0.010	287	0.010	310	0.010	324	0.010
248	0.020	280	0.020	311	0.020	343	0.020	375	0.020	406	0.020	438	0.020	458	0.020
286	0.030	323	0.030	360	0.030	396	0.030	433	0.030	470	0.030	506	0.030	530	0.030
320	0.040	361	0.040	402	0.040	443	0.040	484	0.040	525	0.040	566	0.040	592	0.040
340	0.050	383	0.050	427	0.050	470	0.050	514	0.050	557	0.050	601	0.050	629	0.050
360	0.060	405	0.060	451	0.060	497	0.060	543	0.060	589	0.060	635	0.060	665	0.060
438	0.100	494	0.100	550	0.100	606	0.100	662	0.100	718	0.100	774	0.100	810	0.100
438	0.150	494	0.150	550	0.150	606	0.150	662	0.150	718	0.150	774	0.150	810	0.150

Table 4.17. "k" values found by iteration for API (1993) method

		1 m	2 m	3 m	4 m	5 m	6 m	7 m	8 m	9 m
ITERATION 1	**Long**	20000	20000	20000	20000	20000	20000	20000	20000	20000
	Trans	20000	20000	20000	20000	20000	20000	20000	20000	20000
ITERATION 2	**Long**	17520	19760	22000	24240	26480	28720	30960	32400	32400
	Trans	11040	14351	19497	24240	26480	28720	30960	32400	32400
ITERATION 3	**Long**	17520	19760	22000	24240	26480	28720	30960	32400	32400
	Trans	9975	12805	17220	24240	26480	28720	30960	32400	32400
ITERATION 4	**Long**	17520	19760	22000	24240	26480	28720	30960	32400	32400
	Trans	9674	12348	16485	23383	26480	28720	30960	32400	32400
ITERATION 5	**Long**	17520	19760	22000	24240	26480	28720	30960	32400	32400
	Trans	9564	12180	16210	22905	26480	28720	30960	32400	32400
ITERATION 6	**Long**	17520	19760	22000	24240	26480	28720	30960	32400	32400
	Trans	9520	12112	16099	22710	26480	28720	30960	32400	32400
ITERATION 7	**Long**	17520	19760	22000	24240	26480	28720	30960	32400	32400
	Trans	9502	12085	16053	22631	26480	28720	30960	32400	32400
ITERATION 8	**Long**	17520	19760	22000	24240	26480	28720	30960	32400	32400
	Trans	9495	12073	16035	22599	26480	28720	30960	32400	32400

Table 4.18. Design parameters from iterative response spectrum analysis in clay for method of API (1993)

	M_{long} (kN.m)	M_{trans} (kN.m)	δ_{long} (m)	δ_{trans} (m)
ITERATION 1	15205	23450	0.011	0.009
ITERATION 2	15200	23232	0.011	0.010
ITERATION 3	15200	23165	0.011	0.011
ITERATION 4	15200	23140	0.011	0.011
ITERATION 5	15200	23129	0.011	0.011
ITERATION 6	15200	23125	0.011	0.011
ITERATION 7	15200	23123	0.011	0.011
ITERATION 8	15200	23123	0.011	0.011

4.3.1.2.3 Method of American Petroleum Institute (1993)

Another procedure for the computation of p-y curves in clayey soils was recommended by API (1993). In fact the procedure was very similar to the method of Matlock (1970) except from the definition of the parabolic section of curves. Matlock had used a formulation to define the initial parabolic section of the p-y curve. However, API recommended some constants for the computation of the parabolic portion of the curve. A spreadsheet solution in Excel was performed for time efficiency as summarized in Table 4.15.

4.3.1.2.3 Proposed Simple Method

Similar to the case of sands, simple method proposed in this study can be claimed to be economical when computation and modelling effort is considered. The parameters that would be used for the computation of the p-y curve were selected according to the soil type, lateral loading and group efficiency. Soil type factor was selected as 0.85 according to the recommendation in Chapter 3 and for seismic loading no group reduction was considered. The factor for depth of fixity was calculated after each step using the lateral load values at the top of the pile. Effective moment of inertia was assumed to be the half of the gross value. The calculation process of 1st step was presented as an example. The remaining steps were done in the same way and the calculated spring constants were summarized in Table 4.19.

1st iteration step:

To begin the iteration, a stiffness value of 20000 kN/m is assigned to the springs.

From the analysis results:

V_x = 478 kN

V_y = 1419 kN

Using Equation 3.8;

$\alpha_x = 0.65*\ln(478)-0.10 = 3.910$

$\alpha_y = 0.65*\ln(1419)-0.10 = 4.618$

Using Equation 3.5;

$$y_x = \left(\frac{478 \times 3.910^3 \times 1^3}{12 \times 24800000 \times \left(\frac{\pi \times 1^4}{64} \right) \times 0.5} \right) \times \frac{0.85}{1.00}$$

$$y_x = 0.0033m$$

$$y_y = \left(\frac{1419 \times 4.618^3 \times 1^3}{12 \times 24800000 \times \left(\frac{\pi \times 1^4}{64} \right) \times 0.5} \right) \times \frac{0.85}{1.00}$$

$$y_y = 0.016m$$

Spring constants for next iteration step:

$$k_x = \frac{V_x}{y_x} = \frac{478}{0.0033} = 144848kN/m$$

$$k_y = \frac{V_y}{y_y} = \frac{1419}{0.016} = 88688kN/m$$

Table 4.19. Calculated spring constants at each step by simple method for clay

	LONG. DIRECTION SPRING	TRANS. DIRECTION SPRING
ITERATION 1	20000	20000
ITERATION 2	144848	88688
ITERATION 3	137153	86377
ITERATION 4	136875	86327
ITERATION 5	136866	86345

Table 4.20. Design parameters found from iterative response spectrum analysis in clay for simple method

	M_{long} (kN.m)	M_{trans} (kN.m)	δ_{long} (m)	δ_{trans} (m)
ITERATION 1	14849	21787	0.025	0.022
ITERATION 2	15435	24058	0.004	0.005
ITERATION 3	15431	24054	0.005	0.006
ITERATION 4	15431	24054	0.005	0.006
ITERATION 5	15431	24054	0.005	0.006

4.3.2 Response Spectrum Analysis for Poor Soil and Rigid Pier Conditions

In addition to the bridge model with flexible pier in competent soil, three different cases were also investigated for comparison.

1. Rigid pier (Rectangular 3m x 1m) – Competent soil
2. Flexible pier (Circular 1.5 m) – Poor soil
3. Rigid pier (Rectangular 3m x 1m) – Poor soil

Lastly, the bridge model is analyzed by assigning a fix support at the foundation level to identify significance of the soil-structure interaction on

Table 4.21. Summary of response spectrum analysis for different soil types and different column stiffnesses

			SANDY SOIL			CLAYEY SOIL			
			Reese	API	Simple	Matlock	API	Simple	Fix
COMPETENT SOIL	FLEXIBLE COLUMN	$M_{LONG.}$	15300	15344	15449	15314	15200	15431	16167
		$M_{TRAN.}$	23690	23882	24123	23331	21123	24054	25749
		$\delta_{LONG.}$	0.006	0.006	0.004	0.007	0.011	0.005	----
		$\delta_{TRAN.}$	0.006	0.005	0.005	0.009	0.011	0.006	----
	RIGID COLUMN	$M_{LONG.}$	21340	21365	21529	21273	21140	21499	22988
		$M_{TRAN.}$	39432	39681	40798	38793	38556	40823	48303
		$\delta_{LONG.}$	0.008	0.008	0.005	0.010	0.014	0.006	----
		$\delta_{TRAN.}$	0.009	0.007	0.006	0.014	0.016	0.007	----
POOR SOIL	FLEXIBLE COLUMN	$M_{LONG.}$	15284	15144	15424	14959	14589	15418	16167
		$M_{TRAN.}$	23545	23283	24029	21079	20899	23994	25749
		$\delta_{LONG.}$	0.006	0.011	0.005	0.020	0.030	0.005	----
		$\delta_{TRAN.}$	0.008	0.010	0.006	0.024	0.025	0.006	----
	RIGID COLUMN	$M_{LONG.}$	21311	21126	21489	20408	19863	21480	22988
		$M_{TRAN.}$	39412	38890	40822	31597	31159	40816	48303
		$\delta_{LONG.}$	0.009	0.014	0.007	0.029	0.039	0.007	----
		$\delta_{TRAN.}$	0.007	0.014	0.008	0.031	0.032	0.008	----

Table 4.22. Dynamic characteristics of sample bridge for different methods

ANALYSIS #	SOIL TYPE	COLUMN TYPE	ANALYSIS METHOD	FUNDAMENTAL PERIOD	
				LONG.	TRANS.
1	Competent Sand	Flexible	Reese	1.26	0.71
2	Competent Sand	Flexible	API	1.26	0.70
3	Competent Sand	Flexible	Simple	1.26	0.69
4	Competent Sand	Rigid	Reese	1.19	0.51
5	Competent Sand	Rigid	API	1.19	0.49
6	Competent Sand	Rigid	Simple	1.19	0.47
7	Competent Clay	Flexible	Matlock	1.26	0.73
8	Competent Clay	Flexible	API	1.27	0.75
9	Competent Clay	Flexible	Simple	1.26	0.70
10	Competent Clay	Rigid	Matlock	1.20	0.56
11	Competent Clay	Rigid	API	1.20	0.58
12	Competent Clay	Rigid	Simple	1.19	0.49
13	Poor Sand	Flexible	Reese	1.26	0.72
14	Poor Sand	Flexible	API	1.27	0.74
15	Poor Sand	Flexible	Simple	1.26	0.70
16	Poor Sand	Rigid	Reese	1.20	0.49
17	Poor Sand	Rigid	API	1.20	0.56
18	Poor Sand	Rigid	Simple	1.19	0.49
19	Poor Clay	Flexible	Matlock	1.27	0.85
20	Poor Clay	Flexible	API	1.28	0.86
21	Poor Clay	Flexible	Simple	1.26	0.70
22	Poor Clay	Rigid	Matlock	1.21	0.73
23	Poor Clay	Rigid	API	1.22	0.74
24	Poor Clay	Rigid	Simple	1.19	0.49
25	Fix Base	Flexible	----	1.25	0.63
26	Fix Base	Rigid	----	1.17	0.30

structural response. In the computations, competent soil was taken as N=20 for sands and N=15 for clays. Poor soils were assumed to be N=10 for sands and N=5 for clays based on Caltrans Seismic Design Criteria. Analysis results were summarized in terms of bending moments of the column and the lateral deflections of the pile cap in Table 4.21. Fundamental period values in two directions were used for comparison of the differences in dynamic characteristics (Table 4.22).

4.3.3 Summary of Response Spectrum Analysis

Results of the analyses showed that, soil structure interaction is not effective on the response of a bridge with flexible columns. There was 10% difference between results of fix-supported condition and the results of advanced pile model. In this case, column was behaving more like a seismic isolator and the foundation system was not responding to seismic forces. Therefore, SSI could be ignored and a fixed boundary condition could be assigned in analysis.

Soil-structure interaction was more effective on seismic response of structure in clay than a structure in sand with poor soil condition. Bending moments of the column for two cases showed that there is about 25% difference due to SSI.

Simple method results yielded higher column moments compared to results of advanced methods for a bridge with rigid pier at poor clay site. A possible reason for this difference was shifting of the period of bridge as summarized in Table 4.22. Rotation of the pile cap and settlement of piles are important factors affecting column moments. Simple method is based on the assumption that all the rotation of the pile cap is due to the differential axial deformation of piles. However, additional rotation might be introduced to the system due to the settlement of piles. As a result, column moments for simple method were higher compared to other methods especially in poor clays. Some empirical reduction factors might be used for representation of real behavior in compressible soils.

The simple method was not able to define the bending moment of the pile since no pile is modeled in the structural analysis software. However, based on full scale testing, a formulation regarding the depth of fixity and soil resistance factor can be established to define the bending moment of the pile. Unfortunately, most of the test data used in the back calculation of the simple method do not include much information about the bending moment of the pile. The focus of this paper is not to assess the pile moments.

Predicting displacements at pile cap level using simple method has its own setbacks due to the approximations used in this method. Advanced methods in the literature can even conflict with each other in terms of lateral displacement. It is believed that assessing a reliable displacement is a very complex computation in any method.

Response of a 4-span bridge was also investigated for comparison of seismic responses with 2-span bridge. Response spectrum analysis was performed for the same cases as in the 2-span bridge model. A brief summary of response spectrum analysis (RSA) results are provided in Table 4.24 Regarding RSA results, same conclusions can be drawn for 4-span bridge as well.

4.4 Nonlinear Time History Analysis

4-span bridge model was also analyzed nonlinearly using three earthquake records. Time history record of 1999 Kocaeli earthquake recorded at Yarımca, İzmit and Düzce stations were used. Characteristics of the earthquake records at these stations are summarized in Table 4.23. Requirements of AASHTO Guide Specifications for Seismic Isolation design were used for time history analysis. As stated in this specification, pairs of horizontal ground motion time history components were selected for three records. 5 percent damped response spectrum of each component was created for each pair of horizontal ground motion. SRSS (Square root of the sum of squares) spectrum was constructed for the two components. Then an ensemble spectrum was formed by taking the average of the SRSS spectra for individual earthquakes. This spectrum was scaled so that it did not fall below the 1.3 times the 5 percent damped design basis spectrum in the range of 0.5 T_{eff} to 1.5 T_{eff}. An average acceleration spectrum for selected earthquake records was plotted as shown in Figure 4.10. In the figure, response spectra curve was also plotted to indicate that the time history record is response spectrum compatible. From RSA results, it can be observed that results of methods in the literature are close to each other. Therefore, only method of Reese et al. and simple method will be used for comparison. For nonlinear time history analysis, nonlinear p-y curves computed using method of Reese et al. are assigned at every 1.0 m depth as bilinear springs with hysteretic behavior. Same type of spring is used for simple method but whole pile was defined by a single spring assigned at foundation level. Vertical springs were also assigned to the joints representing piles to consider the rotation due to axial deformation in the pile. Finally, bridge model with fix base was analyzed.

From the nonlinear time history analysis results (Table 4.25) it can be concluded that soil-structure interaction is less effective for flexible columns as in the case of RSA. Generally, column design moments for simple method were close to the results of Reese method and both of them were less than fix base condition.

Depending on the vertical component of earthquake record, axial force in the column may sometimes overcome the effect of dead load. These effects should be considered also for the design of column since an increase of bending moment might be tolerated by the axial force in the interaction diagram. Therefore, axial forces at the time of maximum moments were also presented in the results for comparison.

Table 4.23. 1999 Kocaeli EQ (Magnitude:7.4) data recorded at different stations (From METU-EERC)

		STATION		
		İZMİT	**YARIMCA**	**DÜZCE**
Site Classification		Rock	Rock	Soil
Distance to fault rupture (km)		4.26	3.28	17.06
Peak Ground Acceleration (g)	**North-South**	0.167	0.322	0.337
	East-West	0.227	0.230	0.383
	Vertical	0.149	0.291	0.480

Table 4.24. RSA results for 4-span bridge

		PIER 1 (N = 6250 kN)							
		COMPETENT SOIL				POOR SOIL			
		FLEXIBLE COLUMN		RIGID COLUMN		FLEXIBLE COLUMN		RIGID COLUMN	
		M_{long}	M_{trans}	M_{long}	M_{trans}	M_{long}	M_{trans}	M_{long}	M_{trans}
SAND	Reese	18689	18885	23408	41214	18679	18793	23398	40418
	API	18689	18981	23406	41796	18600	18760	23329	39949
	Simple	18781	19144	23499	43254	18761	19101	23475	43322
CLAY	Matlock	18671	18667	23399	39028	18294	17300	22618	32217
	API	18613	18636	23358	38639	17980	17292	22302	32194
	Simple	18764	19112	23491	43066	18126	17984	23486	42933
FIX BASE		19404	20117	24462	48708	19404	20117	24462	48708

		PIER 2 (N = 6050 kN)							
		COMPETENT SOIL				POOR SOIL			
		FLEXIBLE COLUMN		RIGID COLUMN		FLEXIBLE COLUMN		RIGID COLUMN	
		M_{long}	M_{trans}	M_{long}	M_{trans}	M_{long}	M_{trans}	M_{long}	M_{trans}
SAND	Reese	26021	26745	32192	58454	26047	26703	32159	57795
	API	26122	26812	32260	59227	25979	26511	32136	56701
	Simple	26221	27053	32357	61074	26200	27014	32302	61097
CLAY	Matlock	26009	26634	32100	56498	25243	25065	30893	46480
	API	25983	26508	32093	55727	25158	24826	30815	46019
	Simple	26206	27024	32341	60900	25364	25967	32329	60771
FIX BASE		27102	28360	33508	67544	27102	28360	33508	67544

		PIER 3 (N = 6250 kN)							
		COMPETENT SOIL				POOR SOIL			
		FLEXIBLE COLUMN		RIGID COLUMN		FLEXIBLE COLUMN		RIGID COLUMN	
		M_{long}	M_{trans}	M_{long}	M_{trans}	M_{long}	M_{trans}	M_{long}	M_{trans}
SAND	Reese	18687	18865	23405	41327	18676	18778	23395	40645
	API	18686	18955	23403	41869	18597	18734	23326	40040
	Simple	18778	19111	23496	43163	18758	19068	23477	43253
CLAY	Matlock	18668	18660	23395	39332	18295	17312	22616	32490
	API	18610	18621	23356	38856	17978	17284	22300	32370
	Simple	18761	19079	23488	42974	18124	17979	23484	42840
FIX BASE		19401	20083	24460	48600	19401	20083	24460	48600

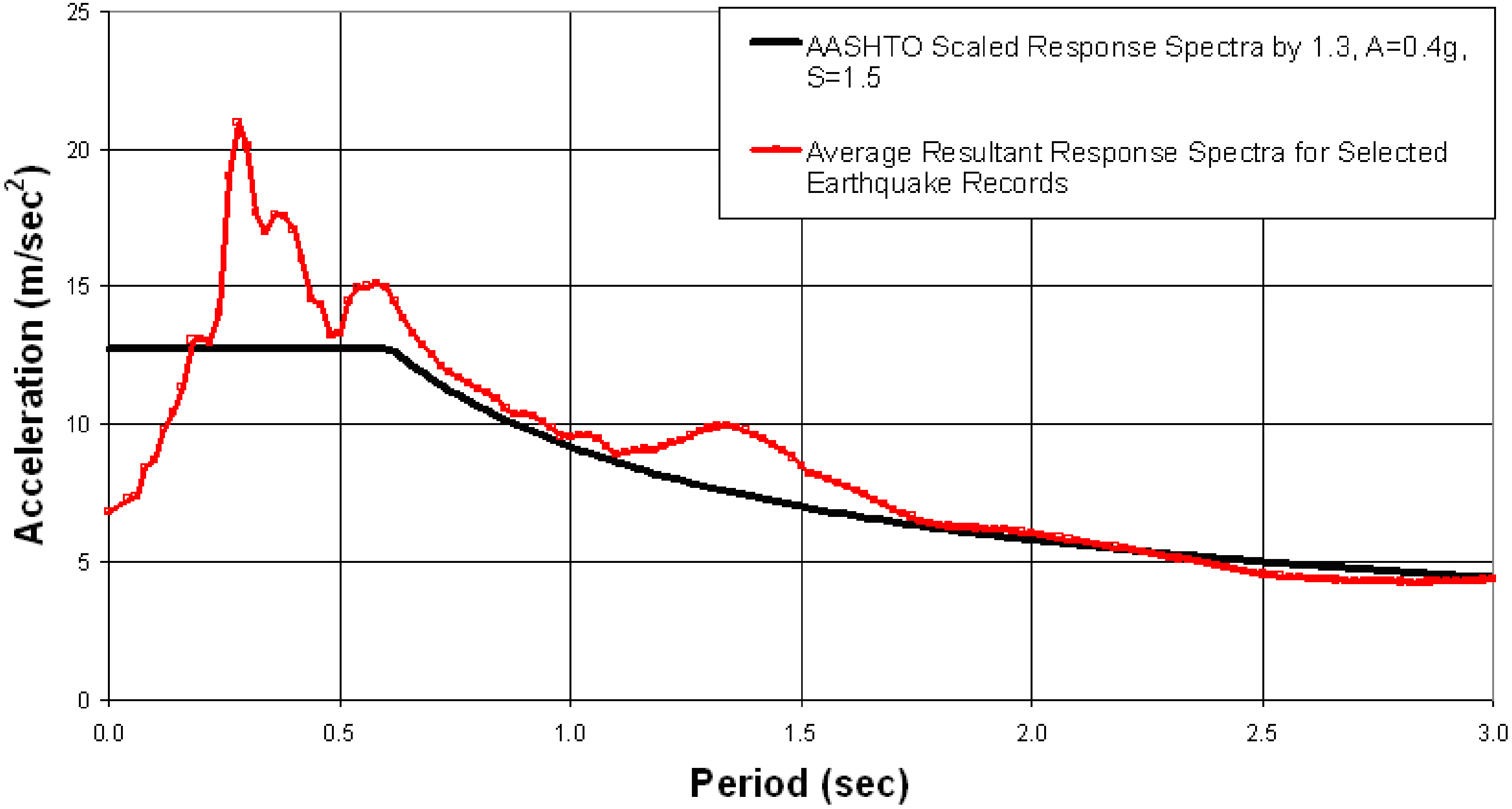


Figure 4.10. Acceleration spectrum for nonlinear time history analysis

Table 4.25. Summary of nonlinear time history analyses

DUZCE RECORD

ANALYSIS METHOD	COLUMN TYPE	SOIL TYPE	DESIGN PARAMETERS FOR PIER 2					
			M_{long}	N_{long}	M_{trans}	N_{trans}	δ_{long}	δ_{trans}
Reese	Flexible	Competent	15499	1034	36698	-34	0.008	0.045
Reese	Flexible	Poor	15440	1238	35743	65	0.008	0.067
Reese	Rigid	Competent	19761	955	59558	22	0.01	0.048
Reese	Rigid	Poor	19854	734	49844	-202	0.011	0.077
Simple	Flexible	Competent	15238	1028	36305	-84	0.007	0.026
Simple	Flexible	Poor	15227	1009	36312	-13	0.007	0.032
Simple	Rigid	Competent	19349	1381	68419	1246	0.007	0.035
Simple	Rigid	Poor	19409	1286	64438	855	0.008	0.042
Fix Base	Flexible	----	15469	713	37204	-218	0	0
Fix Base	Rigid	----	18896	366	89870	227	0	0

IZMIT RECORD

ANALYSIS METHOD	COLUMN TYPE	SOIL TYPE	DESIGN PARAMETERS FOR PIER 2					
			M_{long}	N_{long}	M_{trans}	N_{trans}	δ_{long}	δ_{trans}
Reese	Flexible	Competent	13977	691	20899	2617	0.008	0.024
Reese	Flexible	Poor	14014	951	20031	2608	0.008	0.025
Reese	Rigid	Competent	18108	570	36107	2187	0.009	0.028
Reese	Rigid	Poor	18056	585	35319	1711	0.009	0.031
Simple	Flexible	Competent	13951	175	21141	31	0.004	0.014
Simple	Flexible	Poor	13821	2727	20678	334	0.005	0.018
Simple	Rigid	Competent	18371	-48	38806	2046	0.004	0.018
Simple	Rigid	Poor	18197	-77	34099	1067	0.007	0.020
Fix Base	Flexible	----	14399	3035	22864	-2814	0	0
Fix Base	Rigid	----	19615	-220	56960	-264	0	0

YARIMCA RECORD

ANALYSIS METHOD	COLUMN TYPE	SOIL TYPE	DESIGN PARAMETERS FOR PIER 2					
			M_{long}	N_{long}	M_{trans}	N_{trans}	δ_{long}	δ_{trans}
Reese	Flexible	Competent	27471	4248	24542	-4113	0.042	0.026
Reese	Flexible	Poor	26777	4500	23477	-3999	0.054	0.026
Reese	Rigid	Competent	45683	3139	61364	-709	0.04	0.066
Reese	Rigid	Poor	45374	1230	56046	-325	0.046	0.082
Simple	Flexible	Competent	39954	1445	51259	13480	0.024	0.15
Simple	Flexible	Poor	28099	3857	21268	-3746	0.11	0.12
Simple	Rigid	Competent	47849	5309	72107	1895	0.022	0.03
Simple	Rigid	Poor	47800	5165	72820	2022	0.028	0.038
Fix Base	Flexible	----	42587	3694	41621	303	0	0
Fix Base	Rigid	----	44683	-631	83820	2289	0	0

CHAPTER 5

DISCUSSIONS AND CONCLUSIONS

5.1 Discussions

There are many research conducted to assess the lateral behavior of piles. However, almost no information is provided about the seismic response of the whole structure with proposed lateral behavior of piles. According to the analysis results in this study, pile-soil interaction affects the seismic response of the structure based on the type of soil and the rigidity of the structure.

In this study, a simple method was proposed based on back calculation of full-scale lateral load tests. Since there is not much information available, some assumptions in the method may require checking with test results in the future. Compared with the complicated methods in the literature, this method can be said to be simple and time efficient. No modelling of piles in software is required since total behavior of the pile is represented by a single spring lumped at the foundation level. These single springs have nonlinear characteristics, which can be used both in iterative response spectrum analysis and nonlinear time history analysis. Another advantage of the method is that the calculation of the p-y curve does not require so sophisticated calculations. All the parameters used in the computation are empirical and based on the back calculation of test results. Moreover, no geotechnical parameters from laboratory tests are required since all of them are correlated to SPT-N values. Beam deflection formulas are used and a specific length of the pile is thought as a column fixed at bottom. According to the fixity condition at pile tip, the span length is determined without considering the soil resistance. Following the application of group efficiency factors, the soil resistance factors are back calculated.

Two sample bridge models were used and two types of soil were selected for comparison of the proposed method with the methods in the literature. Using the same soil parameters, p-y curves were calculated as proposed in each method and an iterative response spectrum analysis was performed in LARSA. Column moments and deflection of the pile cap were compared. This process was repeated for 3 different methods both for sands and clays. Analysis were performed both for competent and poor soils in order to check the effect of the soil stiffness on the response of the structure. In addition to that, the bridge column was modeled in two different ways. One with flexible column and the other with rigid column.

Both in the RSA and nonlinear time history analyses, rigidity of the column was the most effective parameter for soil-structure interaction. There was about 10% difference in bending moments for flexible column cases. However, the difference in design moments between the fix based models and the models considering SSI was about 20% for rigid columns. This reduction might be effectively used for the design of the pier.

Response spectrum analyses showed that the lateral resistance of the sand slightly decreased that the column moments were close to each other for competent and poor soils. On the other hand, moments for competent clay were recorded to be 15% greater than the moments for poor clay. Rotation of the pile cap due to settlement of the piles might be the possible reason for this reduction.

As stated in AASHTO Specifications for Seismic Isolation Design, Yarımca record, which was the maximum of three was compared with RSA. Nonlinear time history analysis resulted in more conservative values compared to response spectrum analysis. The differences in bending moment of the column between fix base conditions and models considering SSI were in the range of 30%. Simple method

5.2 Conclusions

- From the analysis results, it could be seen that for seismic design of column of a bridge, proposed method in this study is as effective as the other sophisticated methods in the literature. The single springs define the response of the pile well. On the contrary, for the design of the piles, this simple method was not as effective as other complicated methods since the piles are not modeled in the software.
- Rigidity of the columns is the most dominant factor on lateral response. Soil-structure interaction will take place if the column is rigid enough in the direction of loading. Flexible members behave as seismic isolation system and almost no force is transmitted to the foundation. Since the effect of soil-structure interaction is negligible for such a case, the foundation of the structure can be modeled as a fixed support for simplicity and time efficiency.
- Soil structure interaction is more significant in poor soils than in competent soils. Stiff soils usually have an insignificant impact on the overall dynamic response. Therefore, modelling of piles in a computer solution may be unnecessary depending on soil characteristics. A fix support for foundation will be an easier solution in competent soils. However, the 10% difference between two models can be used for special conditions. Analyses of the response of structures in poor soils require the inclusion of SSI. Pier moments can be reduced by about 30% when response of the soil is considered.
- Settlement of piles has a significant effect on the pier moments. Simple method is unable to define the pile settlement, except inclusion of vertical pile rigidity. Neglecting the pile cap rotation due to differential pile settlement may result in overdesigning of the pier especially in soft clays. As

the soil becomes stiffer, the requirement of correction for settlement diminishes.

5.3 Recommendations for Further Studies

This study includes most of the available lateral load versus deflection data from full-scale tests on reinforced concrete piles. In the future, some improvement can be done on this method but this improvement should be based on performing full-scale tests. Since the number of lateral load tests on piles is rather limited in Turkey, and those available do not include the detailed soil investigation, a more scientific research should be conducted and a database may be formed for further different studies. Apart from measuring the lateral deflection, bending moments may also be checked throughout the test piles.

REFERENCES

AASHTO (1994), "LRFD Bridge Design Specifications." American Association of State Highway and Transportation Officials, Washington D.C.

AASHTO (1998), "Guide Specifications for Seismic Isolation Design." American Association of State Highway and Transportation Officials, Washington D.C.

AASHTO (2002), "Standard Specifications for Highway Bridges." American Association of State Highway and Transportation Officials, Washington D.C.

ACI 318 (1990), "Building Code Requirements for Reinforced Concrete." American Concrete Institute.

API Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms – Working Stress Design, 20th edition, July 1993, Washington DC.

Arsoy S., Prakash S. (2001), "Evaluating group action of piles under lateral loads in sand.", 15th International Conference on Soil Mechanics and Geotechnical Engineering, August 2001, Istanbul, Turkey.

ATC32, Improved Seismic Design Criteria for California Bridges: Provisional Recommendations, Applied Technology Council, California, 1996

Bowles J. E. (1996), Foundation Analysis and Design, McGraw-Hill Book Company

Broms B. (1964), "Lateral resistance of piles in cohesionless soil.", ASCE Jour. Soil Mech. and Found. Div., 90(3), 123-156.

Caltrans Seismic Design Criteria, California Department of Transportation, June 2006, Version 1.4

Duncan M., Evans Jr. L. T., Ooi P. S. K. (1994), "Lateral load analysis of single piles and drilled shafts." ASCE Journal of Geotechnical Engineering, 120(5), 1018-1033.

Dunhum J. W. (1954), "Pile foundations for buildings." Proc. of the ASCE, Jour. SMF Div., Vol. 80, SM 1, Proc. Paper 385, 1-21.

Düzceer, R. (2004). "Sakhalin doğalgaz ve petrol işleme tesisi kazık işleri." Zemin Mekaniği ve Temel Mühendisliği Onuncu Ulusal Kongresi, 16-17 Eylül 2004, İTÜ, İstanbul

Hamilton J. M., Dunnavant T. W. (1993) “Analysis of behaviour of the Tilbrook Grange lateral test pile.” Large Scale Pile Tests in Clay, Thomas Telford, London.

Hetenyi M. (1946), “Beams on Elastic Foundation.” The University of Michigan Press, Ann Arbor, Mich., 100-111.

Hibbeler, R. C., ”Structural Analysis.” Fourth Edition, Prentice Hall, New Jersey

Huang, A.B., Hsueh, C.K, O’Neill M., Chern, S. and Chen, C. (2001), “Effects of construction on laterally loaded pile groups.” ASCE Journal of Geotechnical and Geoenvironmental Engineering, 127(5), 385-397.

Ismael, N. F. and Klym, T. W. (1978), “Behavior of rigid piers in layered cohesive soils.” ASCE Journal of Geotechnical Engineering, 104(8), 1061-1074.

Matlock H. (1970), “Correlations for design of laterally-loaded piles in soft clay.” Paper No. OTC 1204, Proc. 2nd Annual Offshore Tech. Conf., Vol. 1, Houston, TX, 577-594

McVay M., Casper R., Shang T. (1995), “Lateral response of three-row groups in loose to dense sands at 3D and 5D pile spacing.” ASCE Journal of Geotechnical Engineering, 121(5), 436-441.

Mokwa R. L., Duncan M. (2001), “Experimental evaluation of lateral-load resistance of pile caps.” ASCE Journal of Geotechnical and Geoenvironmental Engineering, 127(2), 185-192.

Ooi S. K., Chang B. K. F., and Wang S. (2004), “Simplified lateral load analyses of fixed-head piles and pile groups.” ASCE Journal of Geotechnical and Geoenvironmental Engineering, 130(11), 1140-1151.

Ooi P. S. K., and Duncan M. (1994), “ Lateral load analysis of groups of piles and drilled shafts.” ASCE Journal of Geotechnical Engineering, 120(6), 1034-1050.

Patra N. R., Prabhakar J. P. (2001), “Ultimate lateral resistance of pile groups in sand.” ASCE Journal of Geotechnical and Geoenvironmental Engineering, 127(6), 481-487.

Prakash S., Kumar S. (1996), “Nonlinear lateral pile deflection prediction in sands.” ASCE Journal of Geotechnical Engineering, 122(2), 130-138.

Poulos H. G., Davis E. H. (1980), Pile Foundation Analysis and Design, John Wiley & Sons, New York.

Reese L. C., Cox W. R., and Koop F. D. (1974), “Analysis of laterally loaded piles in sand.” Paper OTC 2080, Proc. Fifth Offshore Tech. Conf., Houston, TX.

Reese L. C., Van Impe W. F. (2001), “Single Piles and Pile Groups under lateral loading.”, A. A. Balkema, Rotterdam, Brookfield.

Rollins K. M., Lane J. D., and Gerber T. M. (2005), “Measured and computed lateral response of a pile group in sand.” ASCE Journal of Geotechnical and Geoenvironmental Engineering, 131(1), 103-114.

Rollins K. M., Peterson K. T., and Weaver T. J. (1998), “Lateral load behavior of full-scale pile group in clay.” ASCE Journal of Geotechnical and Geoenvironmental Engineering, 124(6), 468-478.

Rollins K. M., Snyder J. L., Broderick R. D. (2005), “Static and dynamic lateral response of a 15 pile group.”, International Conference on Soil Mechanics and Geotechnical Engineering, Osaka, Japan, September 12-16, 2005.

Terzaghi K. (1955), "Evaluation of Coefficients of Subgrade Reaction. “Geotechnique, Vol. 5, No. 4, December, pp. 297-326.

Terzaghi and Peck (1948), “Soil Mechanics in Engineering Practice”, John Wiley & Sons, New York.

Zhang L., McVay M., and Lai P. (1999), “Numerical analysis of laterally loaded 3x3 to 7x3 pile groups in sands.” ASCE Journal of Geotechnical and Geoenvironmental Engineering, 125(11), 936-946.

APPENDIX A

AVAILABLE METHODS IN THE LITERATURE FOR CONSTRUCTION OF p-y CURVES

A.1 p-y Curves For Clays

The methods for obtaining the p-y curves for soft and stiff clay are based on full-scale experiment results. For these methods, a detailed site investigation was done in order to determine the soil parameters, especially the undrained shear strength of the clay. Besides, the dimensions and the stiffness of the piles were determined in the most accurate way. The theory during those testing was the development of the p-y curves, which agree well with the test results. Also p-y curves could easily be used for a computer solution of the lateral load behavior of a pile. The tests were done for both the static and cyclic loading cases.

A.1.1 Response of soft clay (Matlock, 1970)

A.1.1.1 Static Case

The procedure is different for static and cyclic cases. For the static case the method is described step by step as illustrated in Figure A-1.

1) The change in the undrained shear strength (c_u) and the unit weight (γ) with depth is determined. Also, the strain corresponding to one-half the

2) maximum principal stress difference (ε_{50}) is obtained. When the stress-strain curves are unavailable, ε_{50} value can be taken from Table A-1.

Table A-1: Representative values of ε_{50} for normally consolidated clays. (Peck et al., 1974)

Consistency of clay	**Average value of kPa**	ε_{50}
Soft	<48	0.02
Medium	48-96	0.01
Stiff	96-192	0.005

3) For a unit length of pile the ultimate soil resistance is calculated by using the smaller of:

$$p_{ult} = \left[3 + \frac{\gamma'}{c_u} z + \frac{J}{b} z\right] c_u b \qquad \text{(Equation A-1)}$$

$$p_{ult} = 9 c_u b \qquad \text{(Equation A-2)}$$

where; γ' = average effective unit weight from ground surface to p-y curve;
z = depth from the ground surface to p-y curve;
c_u = shear strength at depth z;
b = width of pile.
The J value in the formula can be taken as 0.5 for a soft clay and 0.25 for a medium clay based on the experiments done by Matlock (1970).

Ultimate soil resistance value can be computed at any depth where a p-y curve is needed. However, the shear strength and the effective unit weight values should be selected properly according to the depth.

4) The deflection corresponding to one-half the ultimate soil resistance is computed by:

$$y_{50} = 2.5\varepsilon_{50} b \quad \text{(Equation A-3)}$$

5) The relationship between the deflection and lateral resistance of soil is:

$$\frac{p}{p_{ult}} = 0.5\left(\frac{y}{y_{50}}\right)^{1/3} \quad \text{(Equation A-4)}$$

The value of p remains constant beyond $y = 8y_{50}$.

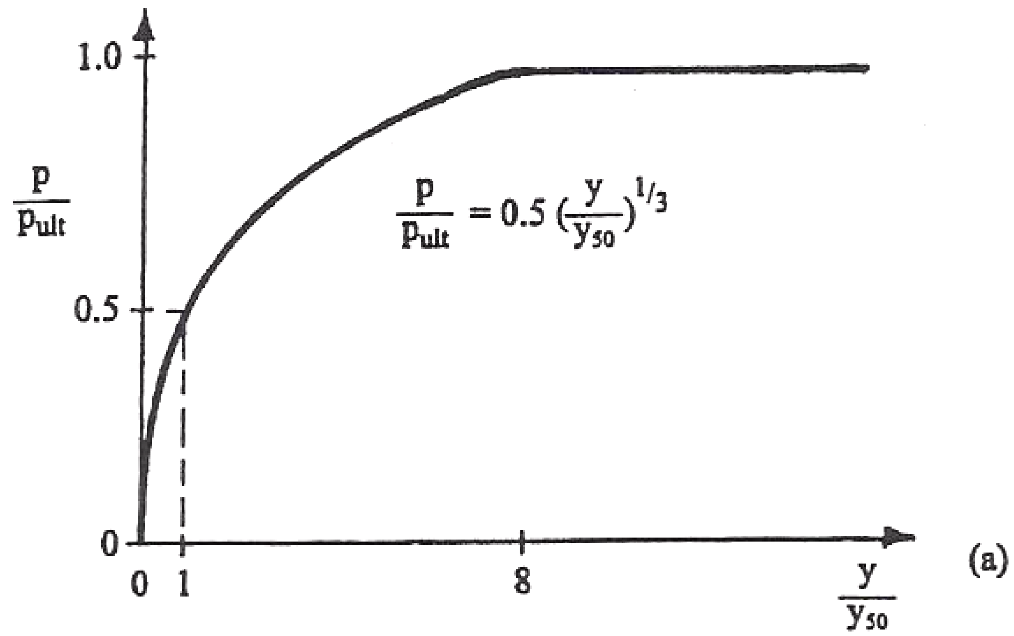


Figure A-1: Illustration of p-y curve for soft clay for static loading (Matlock, 1970)

A.1.1.2 Cyclic Case

For cyclic loading Matlock proposed a step – by – step procedure as illustrated in Figure A-2.

1) p-y curve is constructed similar to the static loading case for p values which are less than $0.72p_u$.
2) Equations to find p_u are solved to find the depth of critical zone.

$$z_r = \frac{6c_u b}{(\gamma' b + Jc_u)} \quad \text{(Equation A-5)}$$

3) When the depth where p-y curve is desired is greater than z_r:

$$p = 0.72 p_{ult} \quad \text{for} \quad y > 3y_{50} \qquad \text{(Equation A-6)}$$

4) When the depth where p-y curve is desired is less than z_r:

$$p = 0.72 p_{ult}\left(\frac{z}{z_r}\right) \quad \text{for} \quad 3y_{50} < y < 15y_{50} \qquad \text{(Equation A-7)}$$

Value of p remains constant after $y = 15y_{50}$

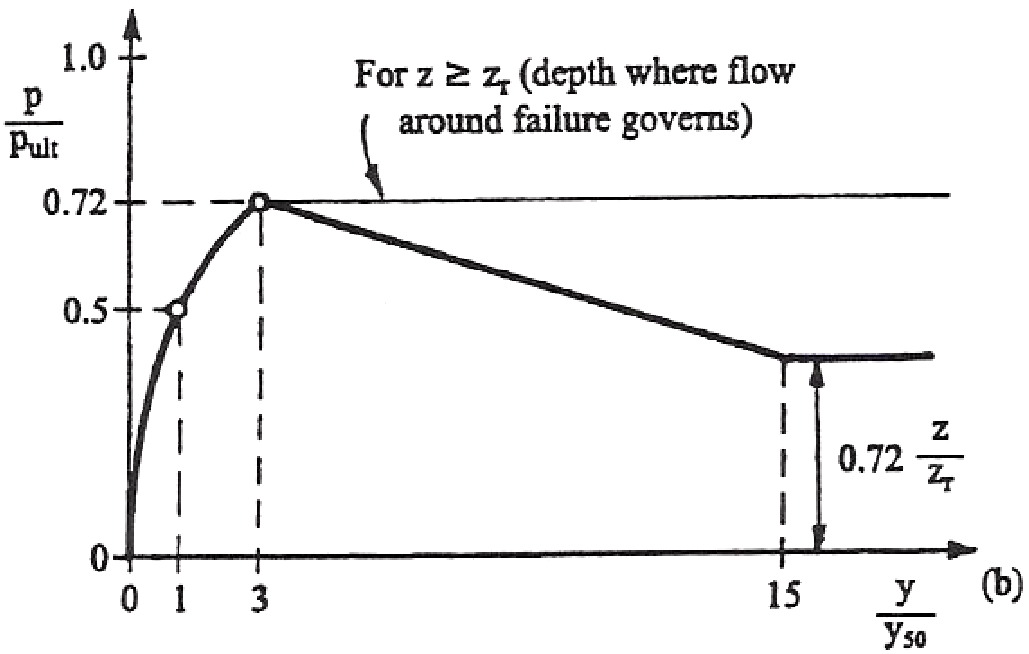


Figure A-2: Illustration of p-y curve for soft clay for cyclic loading (Matlock, 1970)

A.1.2 Response of stiff clay (Reese et al., 1975)

A.1.2.1 Static Case

For static loading conditions, the procedure of computing p-y curves is illustrated in Figure A-3.

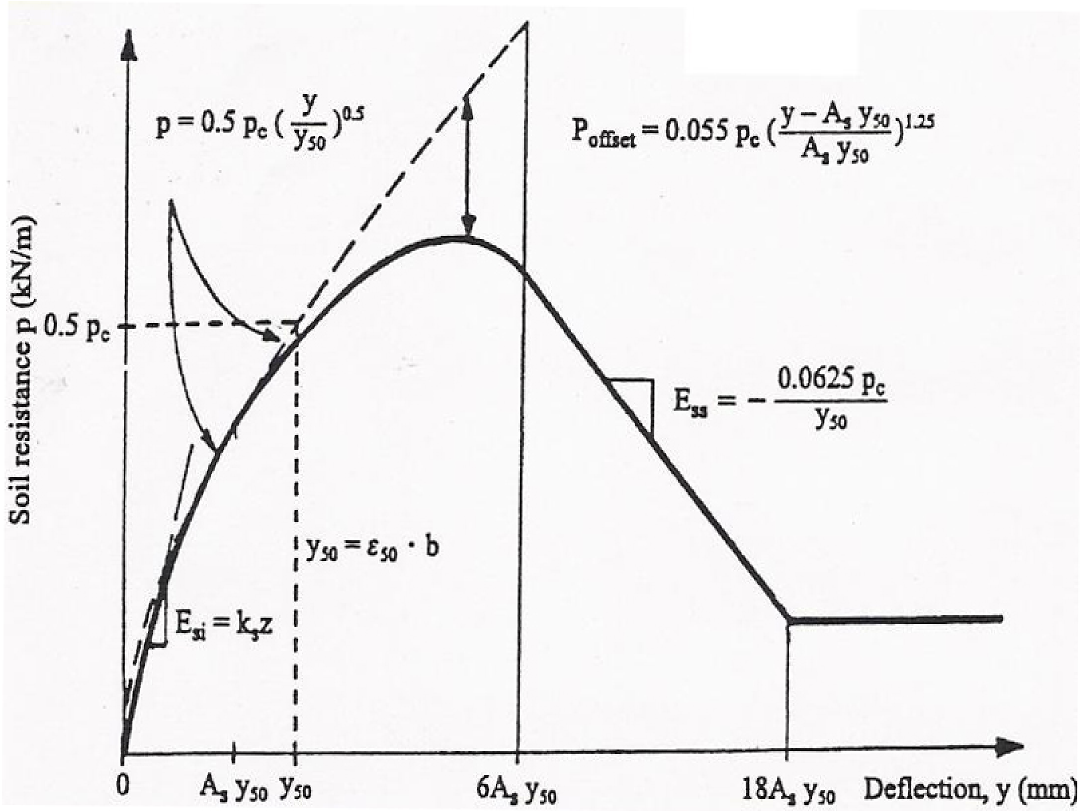


Figure A-3: Illustration of p-y curve for stiff clay for static loading (Reese et al., 1975)

1) Pile diameter, submerged unit weight and undrained shear strength values are determined.
2) An average shear strength value over the depth z should be calculated.
3) The ultimate soil resistance should be computed by using the smaller of:

$$p_{ct} = 2c_a b + \gamma' bz + 2.83c_a z \qquad \text{(Equation A-8)}$$

$$p_{cd} = 11c_u b \qquad \text{(Equation A-9)}$$

4) Non-dimensional depth parameter for static case (A_s) is found from Figure A-4.
5) The initial portion of the p-y curve which is a straight line is drawn as:

$$p = (k_s z)y \qquad \text{(Equation A-10)}$$

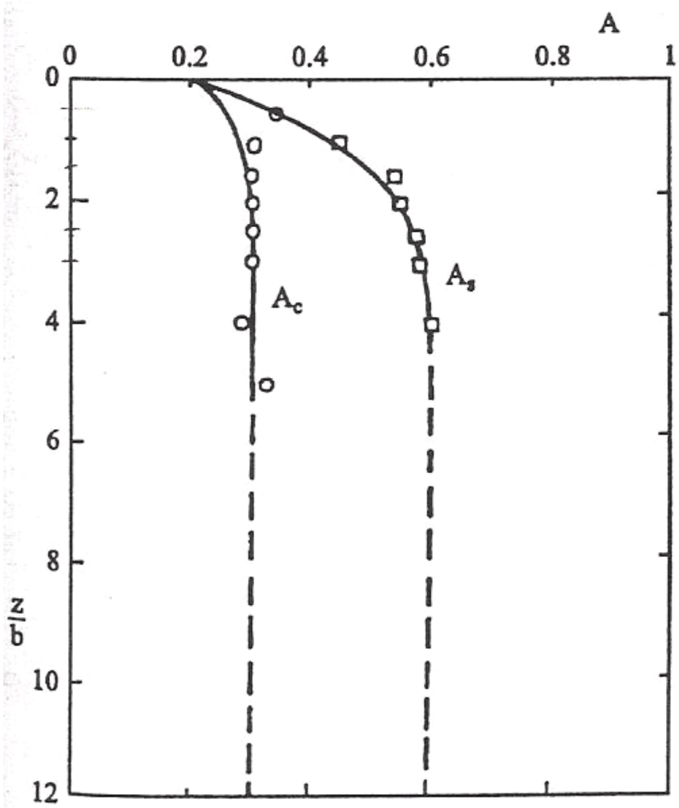


Figure A-4: Values of constants A_s and A_c

For which k_s values can be taken from Table A-2.

Table A-2: Representative values of k_{py} for overconsolidated clays (Reese et al., 1975)

Average undrained shear strength (kPa)			
	50-100	**100-200**	**300-400**
k_{pys} (static) MN/m^3	135	270	540
k_{pyc} (cyclic) MN/m^3	55	110	540

Table A-3: Representative values of ε_{50} for overconsolidated clays. (Reese et al., 1975)

Average undrained shear strength (kPa)			
	50-100	**100-200**	**300-400**
ε_{50}	0.007	0.005	0.004

6) First parabolic portion (from the end of linear part up to $A_s y_{50}$) of the curve can be drawn from the following equation:

$$p = 0.5 p_c \left(\frac{y}{y_{50}} \right)^{0.5} \quad \text{for which} \quad y_{50} = \varepsilon_{50} b \qquad \text{(Equation A-11)}$$

In the absence of laboratory tests ε_{50} may be taken from Table A-3.

7) Second parabolic portion (from $A_s y_{50}$ to 6 $A_s y_{50}$) is a more complex part and calculated as:

$$p = 0.5 p_c \left(\frac{y}{y_{50}} \right)^{0.5} - 0.055 p_c \left(\frac{y - A_s y_{50}}{A_s y_{50}} \right)^{1.25} \qquad \text{(Equation A-12)}$$

8) Next part of the curve is a straight line from 6 $A_s y_{50}$ to 18 $A_s y_{50}$ with a negative slope whose equation is:

$$p = 0.5 p_c (6A_s)^{0.5} - 0.411 p_c - \frac{0.0625}{y_{50}} p_c (y - 6A_s y_{50}) \qquad \text{(Equation A-13)}$$

9) Last step in the formation of p-y curve is the construction of a straight line (for y values larger than $18A_s y_{50}$) which has a constant soil resistance by:

$$p = 0.5 p_c (6A_s)^{0.5} - 0.411 p_c - 0.75 p_c A_s \qquad \text{(Equation A-14)}$$

or from:

$$p = p_c \left(1.225\sqrt{A_s} - 0.75 A_s - 0.411 \right) \qquad \text{(Equation A-15)}$$

This procedure will be applied step by step for the determination of p-y curves in stiff clay. However after the formation of each portion of the curve it should be checked whether the equations intersect with each other in the limiting deformation values. If none of the parts of the curve intersect with the initial straight-line portion, the line itself defines the total p-y curve.

A.1.2.2 Cyclic Case

When the cyclic loading conditions are considered, the first 6 steps of the static case procedure are also valid for the cyclic case except from 4th step.

4) For cyclic loading the non-dimensional parameter A_c can be selected from Figure A-4.

$$yp = 4.1 A_c y_{50} \qquad \text{(Equation A-16)}$$

7) For cyclic loading as illustrated in Figure A-5, there is only one parabolic portion (for y values from the intersection with linear part up to the value of $6y_p$) which is defined as:

$$p = A_c p_c \left[1 - \left| \frac{y - 0.45 y_p}{0.45 y_p} \right|^{0.25} \right] \qquad \text{(Equation A-17)}$$

8) The descending straight line portion (in the range of $0.6y_p<y<1.8y_p$) has an equation as:

$$p = 0.936 A_c p_c - \frac{0.085}{y50} p_c \left(y - 0.6 y_p \right) \qquad \text{(Equation A-18)}$$

9) Final non-sloped straight line portion is found from:

$$p = 0.936 A_c p_c - \frac{0.102}{y_{50}} p_c y_p \qquad \text{for } y>1.8y_p \qquad \text{(Equation A-19)}$$

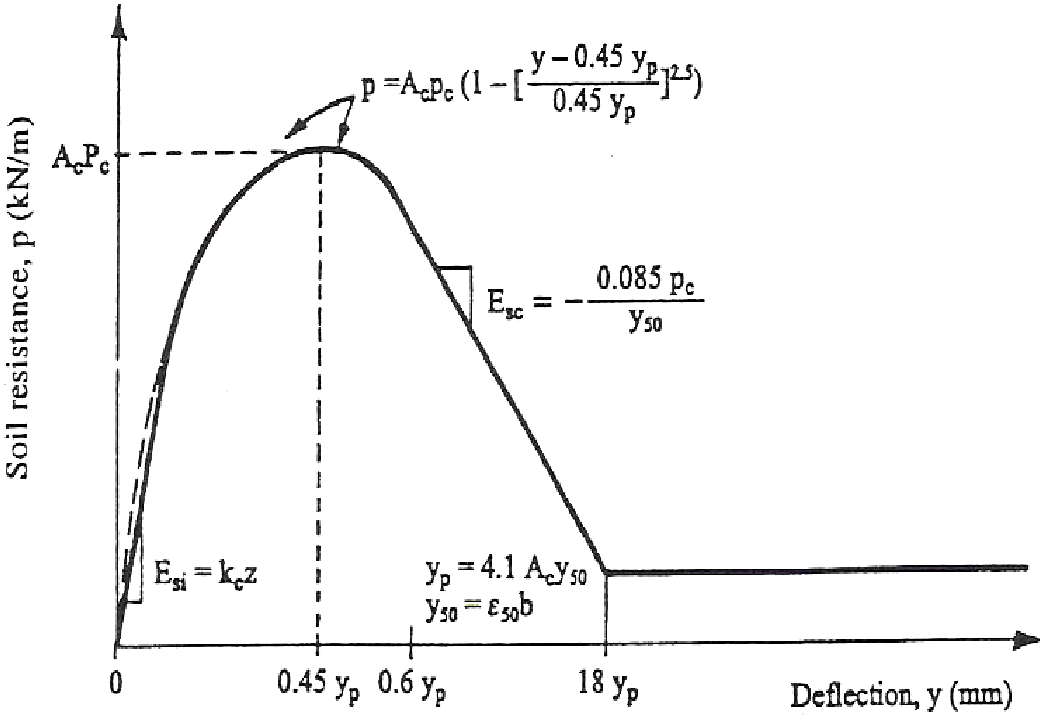


Figure A-5: Illustration of p-y curve for stiff clay for cyclic loading (Reese et al., 1975)

Similar to the static case, the separate portions of the p-y curve for cyclic loading should intersect with each other in the limiting deflection values. If none of them intersect in the limits, the first straight line portion will be the total load-deflection curve.

A.1.3 Response of soft clay (American Petroleum Institute (API), 1993)

Mostly the lateral soil resistance – deflection relationship for piles in soft clay show nonlinear behavior. For short-term static loading in soft clay, the p-y curves can be found from Table A-4.

Table A-4: Load – deflection relationship in soft clay (API, 1993)

p / p_u	y / y_c
0.00	0.0
0.50	1.0
0.72	3.0
1.00	8.0
1.00	∞

where:

p = Actual lateral resistance, kPa

y = Actual lateral deflection, mm

$y_c = 2.5\ \varepsilon_c D$, mm

ε_c = Strain which occurs at one-half the maximum stress on laboratory undrained compression tests of undisturbed soil samples.

In order to calculate the p_u, term which is the ultimate lateral resistance which increases from 3c to 9c, Equations (A-20) and (A-21) should be solved:

$$p_u = 3c + \gamma X + J\frac{cX}{D} \qquad \text{(Equation A-20)}$$

$$p_u = 9c \quad \text{for } X > X_R \qquad \text{(Equation A-21)}$$

Where c is the undrained shear strength of clay, D is the pile diameter, γ is the effective unit weight, J is an empirical constant ranging from 0.25 to 0.50, X is the depth from ground surface and X_R is the depth below soil syrface to the bottom of reduced resistance zone that can be calculated as:

$$X_R = \frac{6D}{\frac{\gamma D}{c} + J} \qquad \text{(Equation A-22)}$$

A.1.4 Response of stiff clay (American Petroleum Institute (API), 1993)

Showing nonlinear stress-strain relationship as soft clays, stiff clays should be considered carefully because of rapid deterioration of load carrying capacity for large deflection values under cyclic loads especially. However, no specific method is proposed for stiff clays.

A.2 p-y Curves For Sands

A.2.1 Static and Cyclic Case (Reese et al., 1974)

In the case of sands, p-y curves can be constructed by the same procedure for static and cyclic loading conditions (Reese et. al. 1974). The detailed procedure is summarized below and illustrated in Figure A-6.

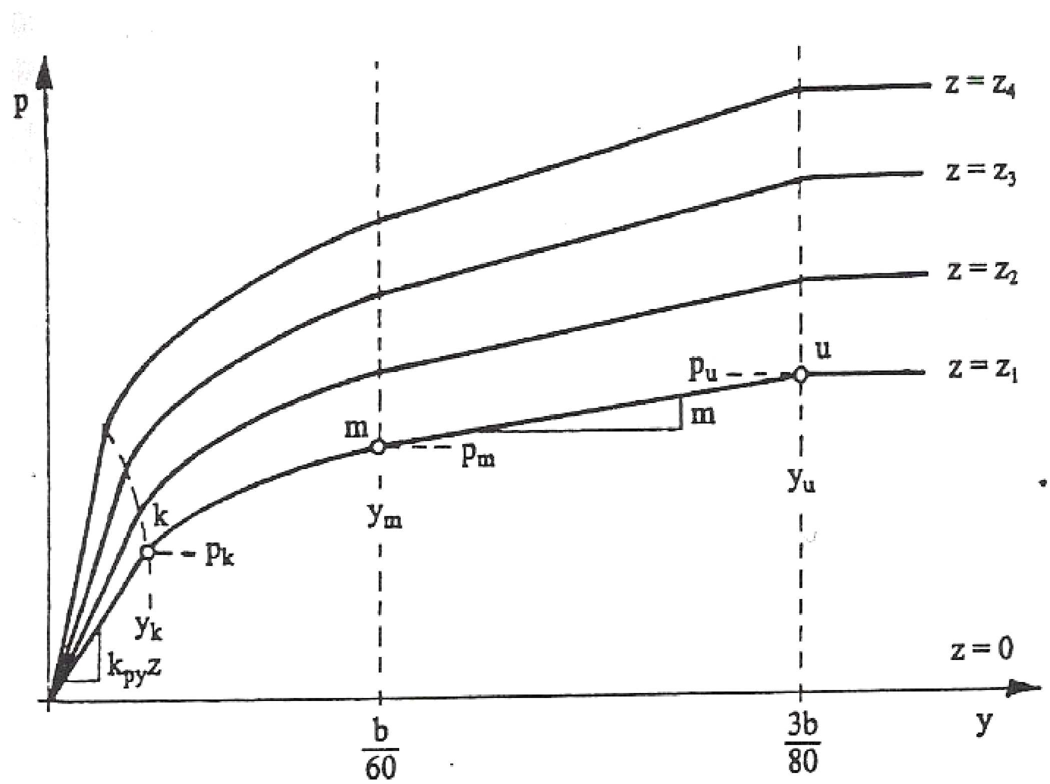


Figure A-6: Illustration of p-y curve for sands (Reese et al., 1974)

1) Pile diameter, friction angle and the unit weight of soil are obtained. For sands below water table, buoyant unit weight is used whereas for sands above water table total unit weight should be included in the equations.

2) Before starting to construct the p-y curve some preliminary computations should be carried out to find out the parameters that will be used in the equations.

$$\alpha = \frac{\phi}{2} \qquad \text{(Equation A-23)}$$

$$\beta = 45 + \frac{\phi}{2} \qquad \text{(Equation A-24)}$$

$$K_0 = 0.4 \qquad \text{(Equation A-25)}$$

$$K_a = \tan^2\left(45 - \frac{\phi}{2}\right) \qquad \text{(Equation A-26)}$$

3) The soil resistance can be set equal to the smaller of p_{st} or p_{sd} values.

$$p_{st} = \gamma z \left[\begin{array}{l} \dfrac{K_0 z \tan\phi \sin\beta}{\tan(\beta - \phi\cos\alpha)} + \dfrac{\tan\beta}{\tan(\beta - \phi)}(b + z\tan\beta\tan\alpha) \\ + K_0 z \tan\beta(\tan\phi\sin\beta - \tan\alpha) - K_a b \end{array} \right] \qquad \text{(Equation A-27)}$$

$$p_{sd} = K_a b \gamma z(\tan^8\beta - 1) + K_0 b \gamma z \tan\phi \tan^4\beta \qquad \text{(Equation A-28)}$$

4) For the computation in previous step, a depth z_t is found, at which there is an intersection between Equations A-27 and A-28. Above this depth Equation A-27 can be used and similarly below this depth Equation A-28 can be used.

5) The depth at which the p-y curve is required is selected and y_u can be taken as 3b/80. For the calculation of ultimate soil resistance, first $\overline{A}_s$ or $\overline{A}_c$ values, which are non-dimensional parameters, are selected from Figure A-7. Next, according to the loading type ultimate resistance of the soil is computed as:

$$p_{ult} = \overline{A}_s p_s \quad \text{or} \quad p_{ult} = \overline{A}_c p_s \qquad \text{(Equation A-29)}$$

After point u (corresponding to y_u and p_{ult}), there will be no change in the load values for increasing deflections.

6) Another point on the p-y curve corresponding to y_m and p_m should be located also. y_m can directly be taken as b/60 and p_m can be calculated by the following equations, after the selection of B_s or B_c values from Figure A-8.

$$p_m = B_s p_s \quad \text{or} \quad p_m = B_c p_s \qquad \text{(Equation A-30)}$$

Between point m and point u, a straight line can be established with the slope of:

$$m = \frac{p_u - p_m}{y_u - y_m} \qquad \text{(Equation A-31)}$$

7) Before the construction of initial linear section, the second portion of the curve which is a parabolic part, should be located by:

$$p = \overline{C} y^{1/n} \qquad \text{(Equation A-32)}$$

where the power of the parabolic section is:

$$n = \frac{p_m}{m y_m} \qquad \text{(Equation A-33)}$$

and the coefficient $\overline{C}$ is:

$$\overline{C} = \frac{p_m}{y_m^{1/n}} \qquad \text{(Equation A-34)}$$

While establishing the parabolic section of the curve, appropriate number of points should be selected to define the curve correctly.

8) The next step will be the formation of the initial straight line portion of the curve using the following equation:

$$p = (k_{py} z) y \qquad \text{(Equation A-35)}$$

where k_{py} values can be taken from Table A-5 or Table A-6.

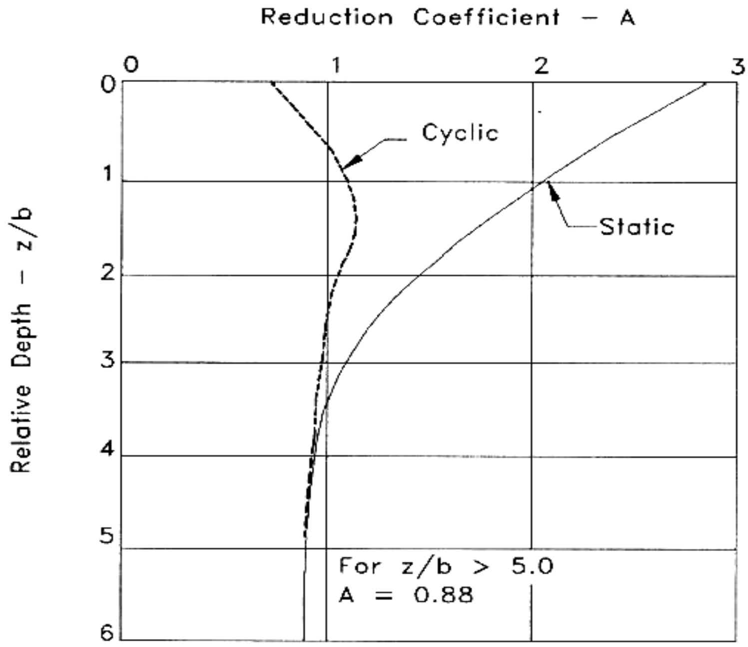


Figure A-7: Values of coefficients A_s and A_c (Reese et al., 1974)

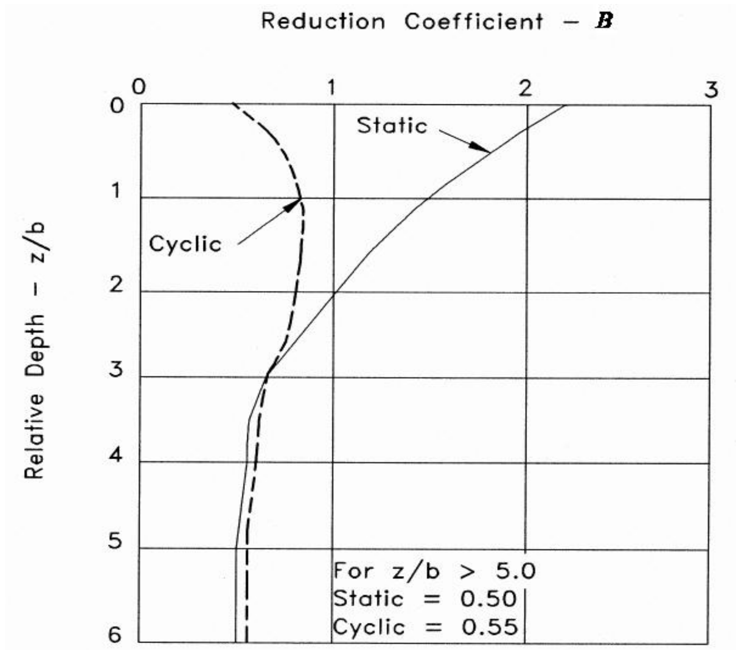


Figure A-8: Coefficient B for soil resistance vs depth (Reese et al., 1974)

Table A-5: Representative values of k_{py} for submerged sand. (Reese et al., 1974)

Relative Density	Loose	Medium	Dense
Recommended k_{py} (MN/m^3)	5.4	16.3	34

Table A-6: Representative values of k_{py} for sand above the water table (Reese et al., 1974)

Relative Density	Loose	Medium	Dense
Recommended k_{py} (MN/m^3)	6.8	24.4	61

The linear part of the curve continues up to a point before the parabolic section starts. In order to locate this limiting point

$$y_k = \left(\frac{\overline{C}}{k_{py} x} \right)^{n/n-1} \qquad \text{(Equation A-36)}$$

The procedure for the formation of p-y curves for sand is based on the assumption of an intersection between the initial straight-line portion and the parabolic portion. However, for some cases there may be no intersection between these curves. For such a case, the initial straight line will define the p-y curve until there is an intersection with another portion.

A.2.2 Response of sand (American Petroleum Institute (API), 1993)

For sands, the lateral soil resistance – deflection relationship is nonlinear and can be approximated for any required depth as:

$$P = A \times p_u \times \tanh\left[\frac{k.H}{A.p_u} .y \right] \qquad \text{(Equation A-37)}$$

where:

A = Factor to account for cyclic or static loading condition.Evaluated by:

$A = 0.9$ for cyclic loading

$A = \left(3.0 - 0.8\frac{H}{D}\right) \geq 0.9$ for static loading

p_u = Ultimate bearing capacity at depth H, kN/m

$$p_{us} = (C_1 \times H + C_2 \times D) \times \gamma \times H \quad \text{(Equation A-38)}$$

$$p_{ud} = C_3 \times D \times \gamma \times H \quad \text{(Equation A-39)}$$

C_1, C_2, C_3 values can be found from Figure A-9.

k = Initial modulus of subgrade reaction, kN/m^3. Determine from Figure 1.1 as function of angle of internal friction, ϕ'

y = Lateral deflection, m

H = Depth, m

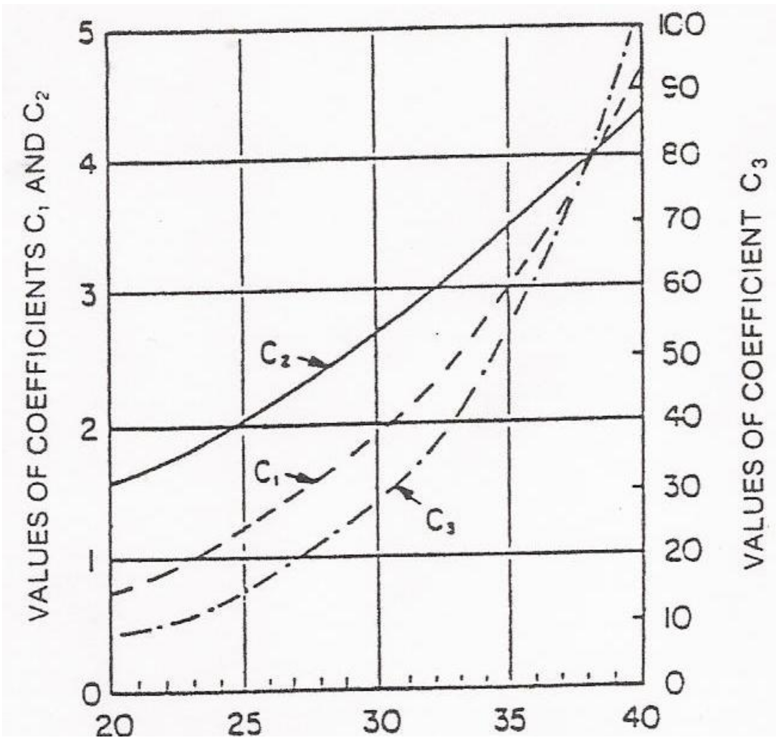


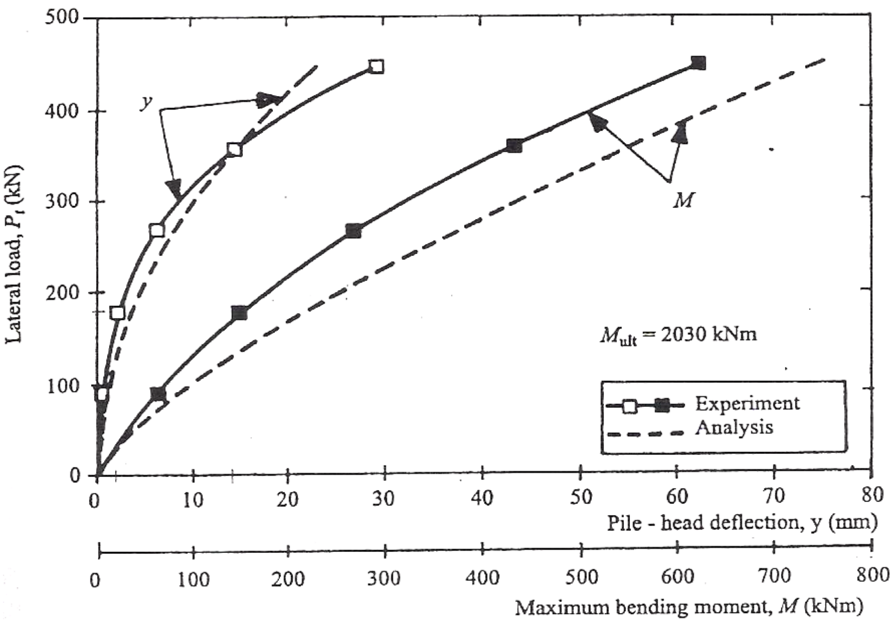
Figure A-9: Empirical constants for computation of ultimate bearin capacity (API, 1993)

APPENDIX B

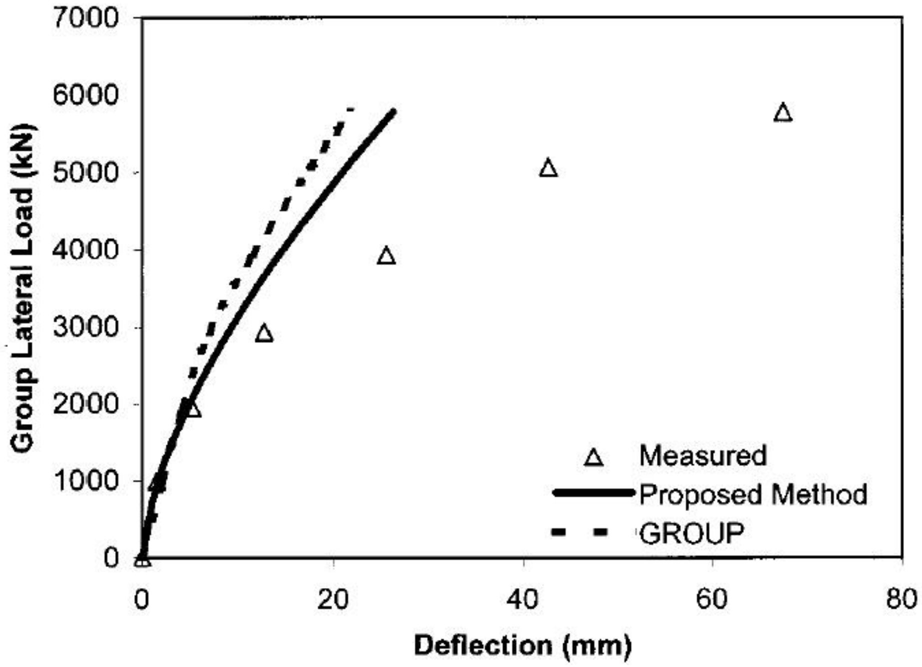
FULL – SCALE LATERAL LOAD TEST RESULTS USED IN THE ANALYSIS

Data used in the back – calculation analysis is from the load – deflection plots of full scale lateral loading tests. The load versus deflection values are given in Table 2.1. For completeness, plots of load-deflection data are shown below in Figures B1 to B10.

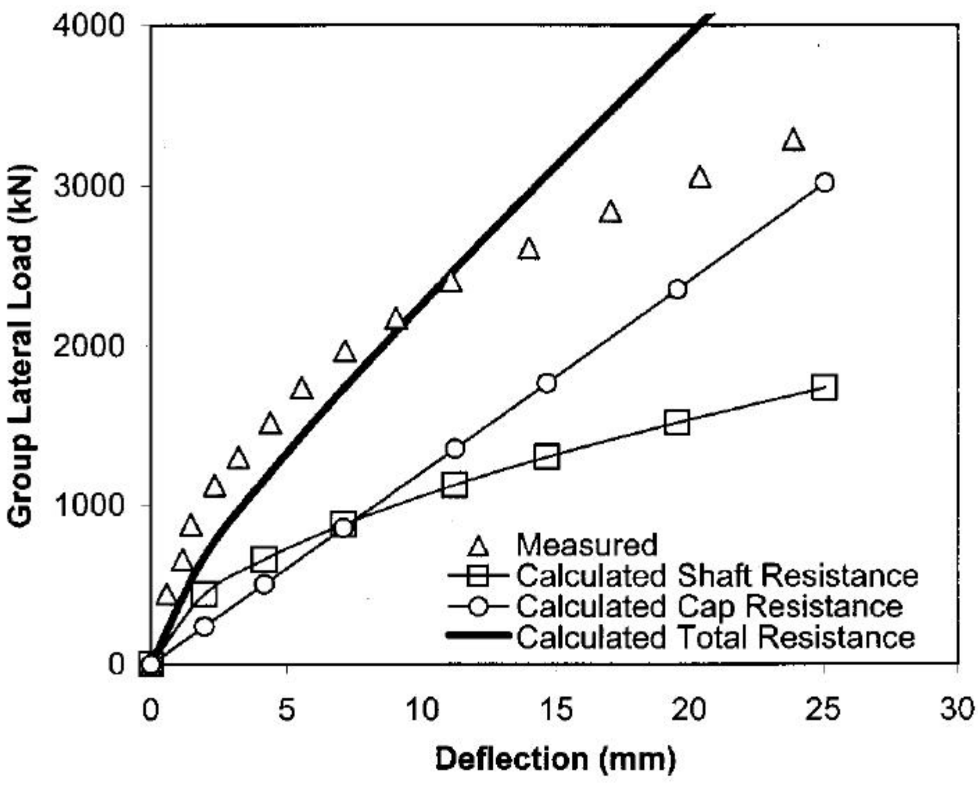
B.1 Houston Test (Reese & Welch, 1975)



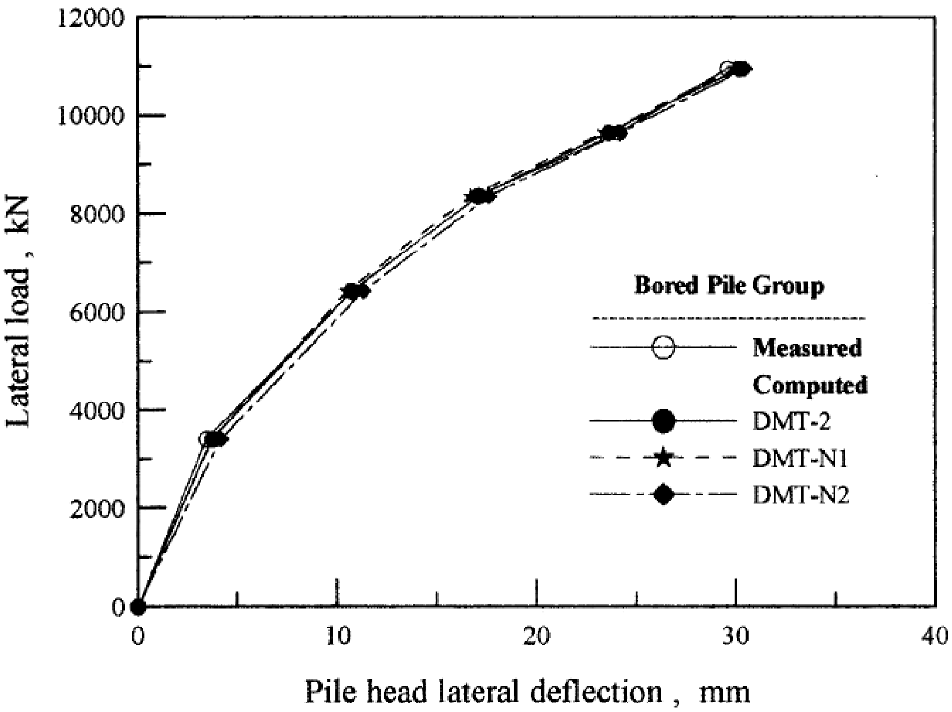
B.2 Hong Kong Test Ng et. al., 2001)



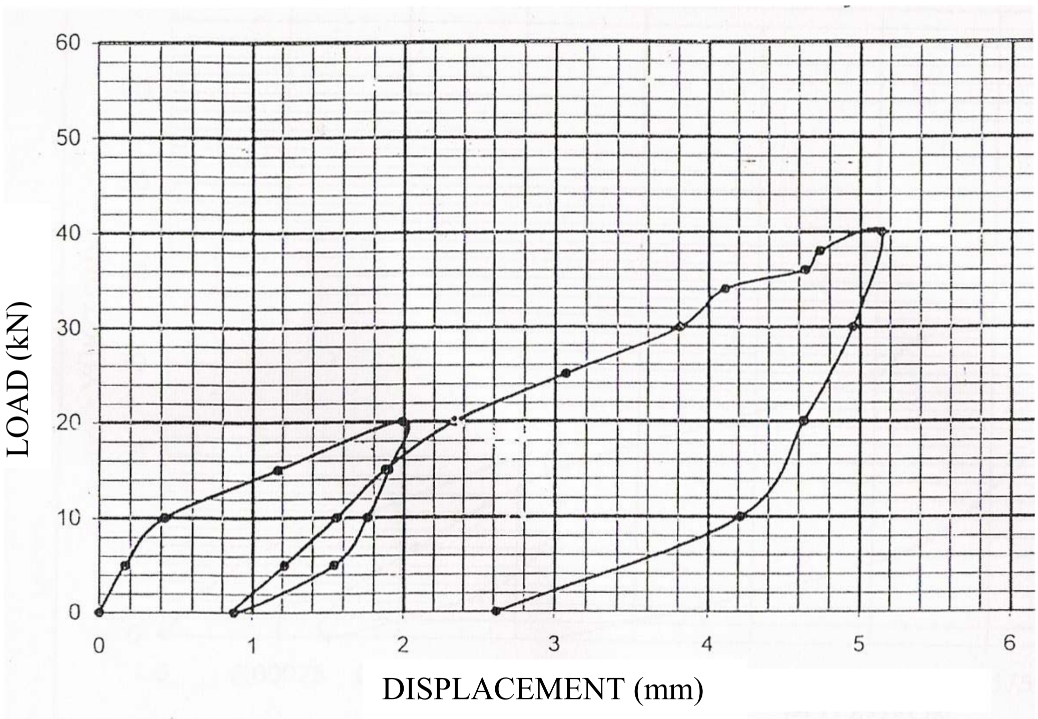
B.3 Las Vegas Test (Zafir & Vanderpool, 1998)



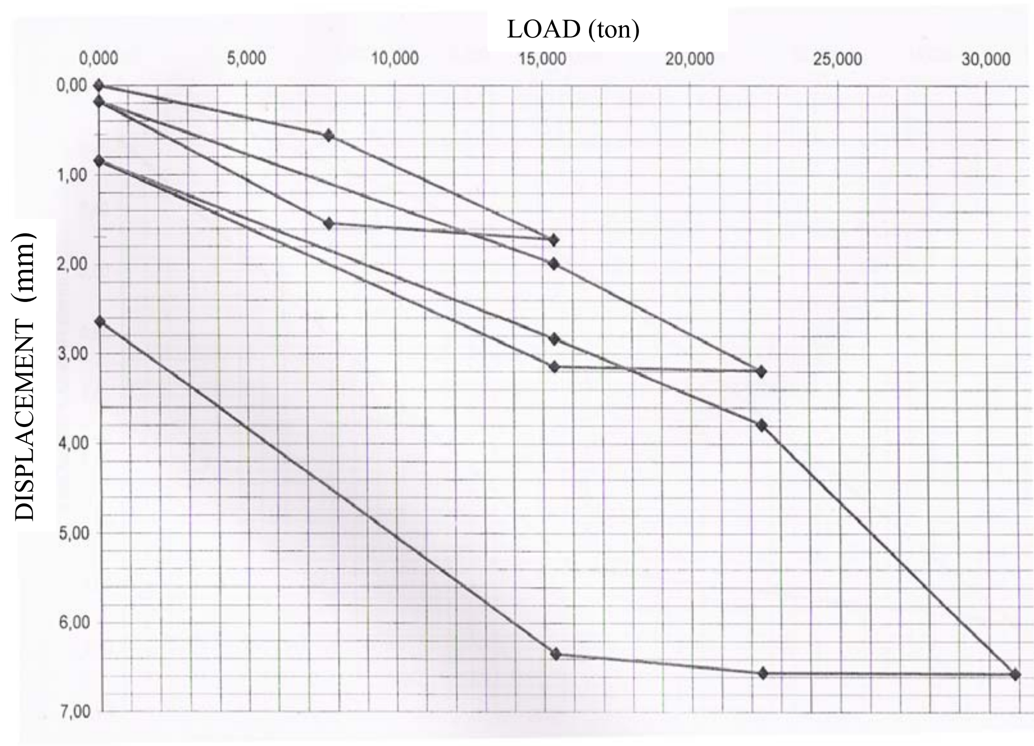
B.4 Taiwan Test (Group Piles) (Huang et. al., 2001)



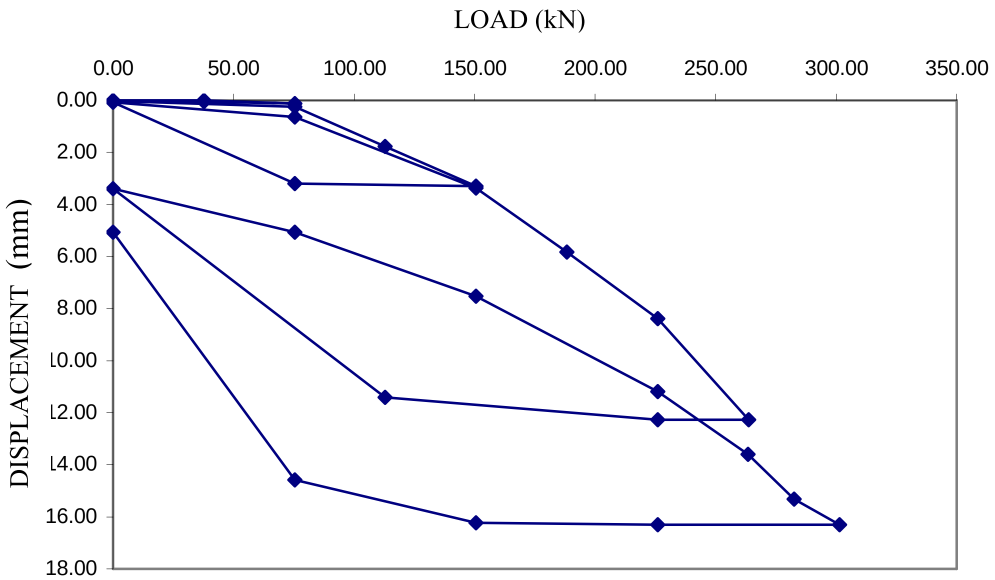
B.5 Trabzon Test (MNG ZEMTAŞ A.Ş., 2002)



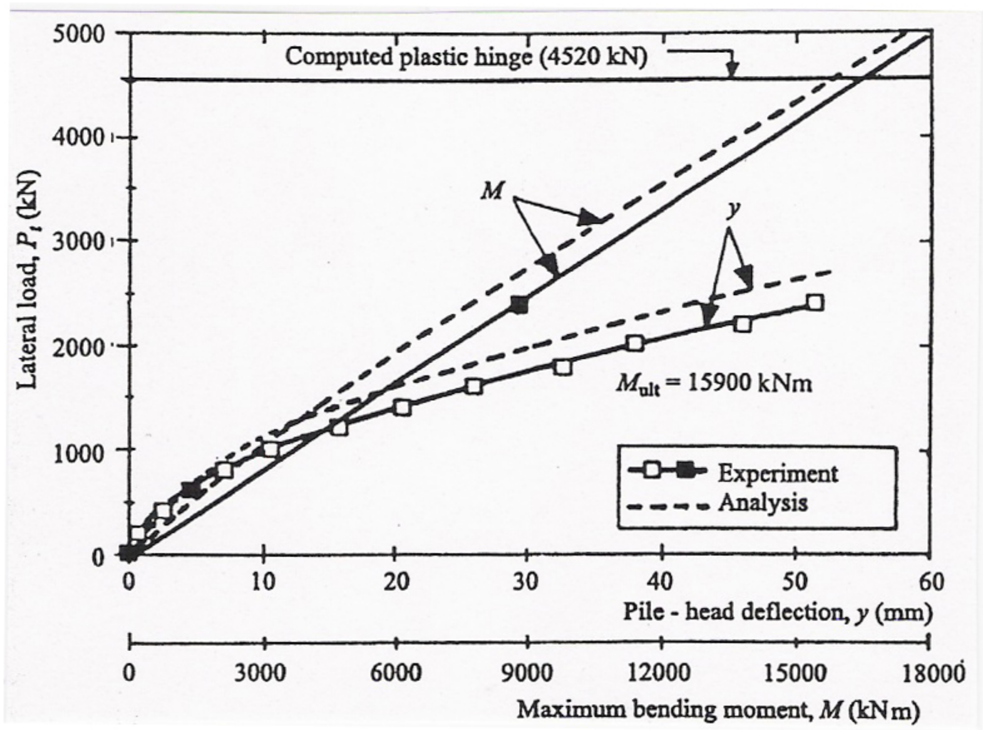
B.6 Mersin Test (Toker Sondaj ve İnş. A.Ş., 2004)



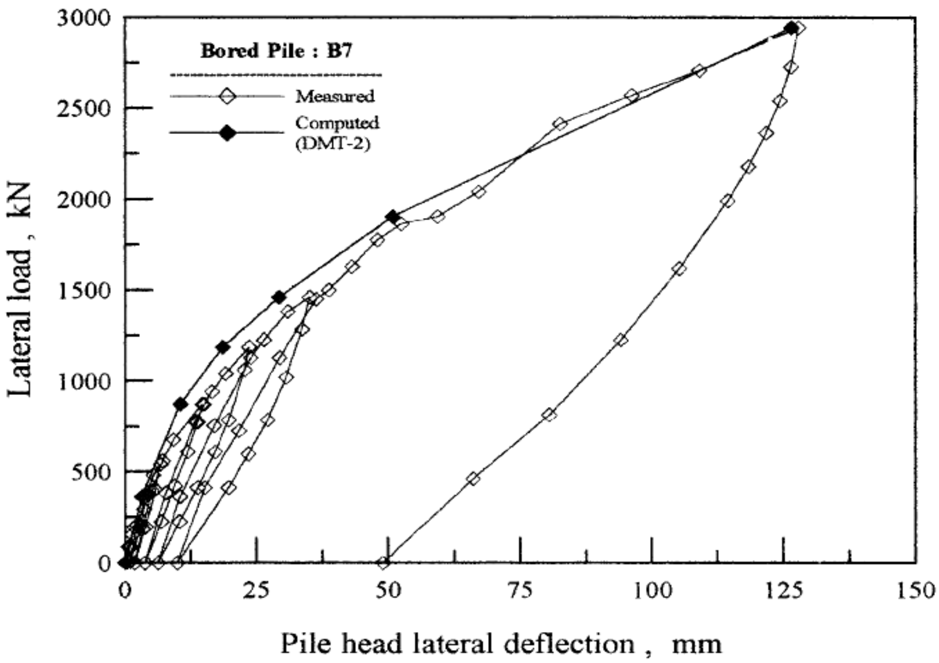
B.7 Çatalağzı Test (Toker Sondaj ve İnş. A.Ş., 2001)



B.8 Garston Test (Price & Wardle, 1987)



B.9 Taiwan Test (Single Pile) (Huang et. al., 2001)



B.10 ---- Test (Bhushan & Lee, 1981)