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SEISMIC BRIDGE PIER ANALYSIS FOR PILE FOUNDATION BY FORCE AND DISPLACEMENT BASED APPROACHES

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Abstract. *Seismic analysis of bridge pier supported on pile foundation requires consideration of soil-pile-structure (kinematic and inertial) interactions. This paper presents the design forces generated for bridge piers with varying height and constant diameter for medium and soft soils in earthquake probability zones considering contribution of soil-pile-structure interactions by developed analytical approaches. The results have shown that the difference in base shear demand between force based and displacement based approach and that between capacity spectrum and displacement based method in general decreases with the increase in slenderness ratio of the pier. The base shear demand by non-linear time history analysis has been found to be much higher compared to that by other methods. The relationship between height and pier cross-section has been developed for different soils and seismic zones such that the base shear demands by force based and displacement based method are of the same order. The overall value of the slenderness ratio works out to be such that failure of the pile shall be as a short column for both medium and soft soil*.

Key words: *Bridge pier pile foundation, Soil-Pile-Structure Interaction, Force based Design, Direct Displacement based Design, Non Linear Time History Method and Capacity Spectrum Method.*

1. INTRODUCTION

Pile foundations are widely used in case of bridges due to very heavy loads of super-structure and/or when adequate bearing capacity of the soil is not available at reasonable depth (1). Such foundations are required to be designed for the lateral seismic load in addition to the gravity loads. The seismic response of pile foundations is greatly influenced by the behavior of the soil into which piles are embedded. However, seismic design codes like NEHRP-97 (2) either ignore the seismic behavior of the piles or greatly simplify the design procedure. This is attributed to foundation flexibility which causes increase in a

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structural time period resulting into reduction of seismic design forces (3). But the studies (4) (5) (6) have shown that this is not always true, neglecting the behavior of foundation piles may prove detrimental. Besides, many theoretical and experimental studies have shown that the design based on rigid foundation assumption is not always secure and hence the dynamic soil-pile-structure interaction need to be considered in the seismic design (7). This has also been recognized by some of the design codes (Eurocode) (8).

The soil-pile-structure interaction involves: (a) kinematic interaction- The seismic shear wave propagating through the soil towards the pile head is modified by the presence of embedded pile due to soil-pile stiffness contrast. It also causes deformation in piles and motion and displacement of pile head. This can occur even in the absence of superstructure. (b) Inertial interaction- The dynamic response of the superstructure induces additional deformation into the pile as well as surrounding soil. Both the effects go on simultaneously (9). Thus the piles are subjected to kinematic forces caused by the deformation of soils by the impinging seismic waves and inertia forces induced near the pile head by structural oscillations (10). Accordingly, the analysis steps will involve: (1) kinematic analysis without superstructure to determine the seismic motion of pile head, which is known as foundation input motion; (2) computation of dynamic stiffness (springs and damping); (3) dynamic analysis of the superstructure supported on springs considering foundation input motion determined above (11). It is desirable that the foundation system should remain elastic while the pier should be detailed for inelastic deformation and energy dissipation. The main reason being: inaccessibility for post seismic inspection, high cost of repair as well as failure of pile before exceeding soil capacity is undesirable (3) (12). The location of pile failure in many cases have been found to be deep from the top (13). Such type of failure is due to soil deformation i.e. kinematic interaction (14). Sometimes engineers choose to design on the basis of inertia force only which is not proper and kinetic interaction must be taken into account (15).

Many researchers have developed approaches for evaluation of impedance function of single pile and use for their dynamic response (16) (17) (18) (19). Similar studies have also been carried out by various researchers for pile groups (20) (21) (22) (23). Effect of spacing of group pile studies made by investigators (Cox et al, Wang et al, Franke, Prakash, Shibata, Schimidt) have been compiled by Antti Larkela (24) and Markis and Gazetas (25). Many other studies on pile foundations have considered the piles as beam on dynamic Winkler foundation and the soil being represented by spring and dampers (9) (14) (15) (26). The non-linear behavior of soil has been used for dynamic response of pile foundation and has been studied by Nogami et al, Tazoh and Shimizu and Holt et al (27) (28) (29) while Kok and Huat, Deziel et al, Guoxi, Kouroussis (30) (31) (32) (33) have applied Finite Element Method for seismic response of bridge piers with pile foundations. The favorable effect of SSI could be exploited to mitigate seismic demands in bridge pier: stiff piers cause hinging in piles near the pile cap which could be avoided by the use of flexible piers (34).

2. Seismic Design of Single Bridge Pier

The work is carried out on a single bridge pier supported on pile foundation (Fig.1) considering soil-pile structure interaction with a focus on the variation in the design output of the methodologies i.e. Force based design, Capacity spectrum method, Direct displacement based design and Non-linear time history method with respect to changes in the height of the pier (6m, 9m, 12m, 15m, 18m). The cross-section of the bridge pier is considered constant i.e.

1.8m. Although the height of the pier depends upon the site condition but as far as possible the slenderness ratio of the pier are kept below 12 (35) so that in case bridge pier fails, the failure is governed by shear rather than flexural. The seismic inertial mass at the top of pier determined from the weight of super structure and weighted live load on the span was calculated as 4277 kN. The foundation consists of the pile cap (5.4m x 5.4m x 1.75m) with 2 x 2 pile group each having cross section of 1.2m diameter and 20m long with spacing of 3.6m c/c. The reinforcement in bridge pier, pile cap and piles are based on code design provisions. The concrete grade M-40 has been used in the pier and M-20 in the pile cap and piles. The reinforcement of grade Fe-415 has been used in all components. Modulus of elasticity for M-40, M-20 and Fe-415 are $3.16\text{x}10^{10}$ N/m^2, $2.2\text{x}10^{10}$ N/m^{2}and $2\text{x}10^{11}$ N/m^2 respectively.

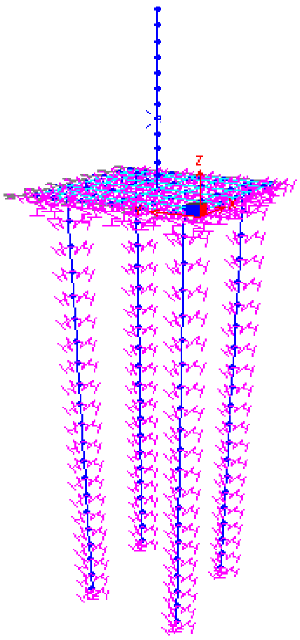


Fig. 1. Bridge pier on pile foundation

The N-values for medium and soft soils were assumed 20 and 10 respectively. The bearing capacity of the soils were computed based on different methods (Ranjan and Rao 2007; IS: 6403-*1981*) (36) (37) and the minimum values 180 kN/m^2 and 100 kN/m^2 have been adopted for medium and soft soils respectively. The subgrade reaction values (k) were determined based on codal provision of IS:2950 part 1 (38). The subgrade reaction in vertical direction obtained as 8.3 x 10^3 kN/m^3 and 4 x 10^3 kN/m^3 for medium and soft soils. Half of these values were considered in the horizontal direction (39). These values were used to determine the spring constants on the pile cap. The vertical subgrade reaction at the pile tips were obtained as 120x10^3 kN/m^3 and 64x10^3 kN/m^3 for medium and soft soils respectively which were used for calculating the spring constants for pile tips. The horizontal modulus of subgrade reaction (η_h) values were adopted as per (IS-2911-part 1) (40) for calculating horizontal spring constants on vertical piles. For medium soil η_h were taken as 7.0 x 10^3 kN/m^3 and 4.75 x 10^3 kN/m^3 for dry and submerged conditions respectively. Taking 8m pile length under submergence as 8m, the average value of η_h were taken as 6.34 x 10^3 kN/m^3. Similarly, the average value of modulus of subgrade reaction for soft soil was determined as 2 x 10^3 kN/m^3.

To consider the effect of soil-pile interaction, the bridge pier along with the piles was modeled as -

1. Pile cap divided into 36 gird elements of size 0.9m x 0.9m. Each element was considered as shell element.
2. Spring constants at each node of the pile cap were calculated as the product of subgrade reaction value and corresponding influence area. The vertical spring constant obtained for the medium and soft soil varied from 1460 kN/m to 6723 kN/m and from 700 kN/m to 3240 kN/m respectively.
3. The pile length was divided into 20 segments with each segment of 20m. Considering triangular distribution of modulus of horizontal reaction, the horizontal spring

constants were determined at each node of the pile. The horizontal spring constant obtained for the medium and soft soil varied from 12.68 x 10^3 kN/m to 133.14 x 10^3 kN/m and from 4 x 10^3 kN/m to 42 x 10^3 kN/m respectively. The vertical spring constant at the pile tip was determined as product of pile cross-section and vertical subgrade reaction at the pile tip. The vertical spring constant obtained for the medium and soft soil is 135.0 x 10^3 kN/m and 72 x 10^3 kN/m respectively.

4. The bridge pier is divided into segments of equal length of 0.5m. Accordingly the bridge pier with different height had different number of segments.

This model was used for analysis by Force Based, capacity spectrum and Nonlinear Time History Analysis methods.

2.1. Force based design method

The Force Based Design (FBD) concept transmutes empirical parameters encompassing appropriate support conditions for calculating member elastic stiffness, spectral acceleration (Sa/g) determination for calculated fundamental time period, presumed damping factor, probability of occurrence of earthquake as zone factor Z, reduction of spectral acceleration by Response reduction factor R for transforming structural elastic behavior into inelastic, Structural importance consideration i.e. Immediate occupancy of structure after earthquake as Importance factor I. In case, the displacements are not within the specified limits the analysis is repeated with the revised member dimensions until drift criteria is satisfied. The Indian code (*IS: 1893 Part1*- 2002)[41], for seismic analysis has adopted force based design that has been considered in this study. The empirical parameters adopted are - Earthquake response spectrum with damping ξ= 5 %, Sa = 0.36g, 0.24g, 0.16g, I = 1.5, R = 4.

2.2. Direct displacement based design

Direct Displacement Based Design (DDBD) method is based on achieving required performance based on defined damage level. This method uses an equivalent single degree of freedom system and energy dissipation capacity is represented as equivalent viscous damping. The procedure of design uses displacement spectra generated for various equivalent damping factors. The displacement at four sec considered as corner period is determined as spectral displacement. The sequential calculation of yield curvature (ϕ_y), yield displacement Δ_y and design displacement Δ_d as given in eqns (1) (2) (4) respectively leads to the determination of the effective time period of the structure. The design displacement is the minimum of displacement obtained from the product of the yield displacement and the ductility or the product of the limiting drift and the height. The effective period is calculated from the target spectral displacement considering equivalent viscous damping as given in eqn (5), which is based on ductility to be achieved. The effective stiffness as given in eqn (6) of the pier is then calculated to determine, the design base shear which is the product of the design displacement and effective stiffness of the pier. For the given diameter (D) and height (h) of the bridge pier.

$$\phi_y = 2.25 \times {}^{\epsilon_y}/_{D} \tag{1}$$

where, ϵ_y= strain in steel.

$$\Delta_y = \phi_y \times \frac{(h+L_{sp})^2}{3} \tag{2}$$

Where, Strain penetration length

$$L_{sp} = 0.22 \times f_{ye} \times d_{bl} \tag{3}$$

$$\Delta_d = \mu \times \Delta_y \text{ limited to } \theta_d \times h \tag{4}$$

Where, μ = assumed displacement ductility = 4, θ_d= limiting rotation (taken as 3.5%).
Drift θ_d at pier base = 0.035 radian,

Equivalent Viscous Damping

$$\xi_{eq}=0.05+0.444((\mu-1)/\mu\,\pi) \tag{5}$$

$$\text{Stiffness} = 4\pi 2m/Te2 \tag{6}$$

To account for soil pile-structure interaction, the horizontal displacements at the pier top were determined considering the base shear values obtained considering bridge pier fixed at the base using Matock and Reese method (Swami Saran) (42). Also, the damping ratio for the foundation was considered as 0.05. The former was used in calculating design displacement and the latter in calculating equivalent damping as detailed in reference (43).

2.3. Capacity spectrum method

The Capacity Spectrum Method (CSM) method the structure is idealized as a single degree of freedom (SDOF) and reduced secant stiffness and increased damping proportional to hysteretic energy are used to estimate the response spectra of non-linear system which represent the inelastic seismic demand. The seismic demand curve was generated based upon the site location and foundation condition (seismic zone and soil type) using design response spectrum on acceleration displacement response spectrum format (called as seismic demand) (ATC 40) (44). The pushover curve was generated by applying step wise incremental load on the top of the bridge pier until failure. The force displacement relationship obtained are based on considered Non-linear M-φ Plastic Hinge relation of cross-section, Takeda Hysteretic model, material stress strain relationship, plastic hinge length and steel yield stress. The performance level of the structure is the point of interaction of seismic demand and capacity curves plotted on acceleration displacement response spectrum format. In case of capacity spectrum method the design displacement is determined from the intersection of the capacity and the demand spectrum. From the capacity spectrum method, base shear, yield displacement, design displacement, ductility, effective period and equivalent viscous damping is obtained.

2.4. Nonlinear time history method

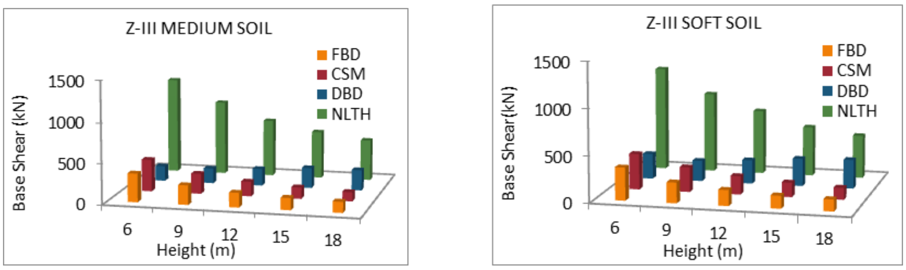
The inelastic time history analysis is considered as the most sophisticated tool of analysis and is often considered as bench mark for the comparison of responses with other inelastic methods. In this method, the seismic response and displacement are determined directly through non-linear dynamic analysis using ground motion histories that are actual recorded earthquake motions, leading to the responses which are sensitive to the individual ground motion. It is assumed that the mathematical model takes into account the effects of material inelastic response and the calculated internal forces are reasonable approximation of those expected during earthquake. The time history analysis consists of direct step by step integration of the equation of motion of the mathematical model of a structure. The various inelastic component properties considered are yield strength, post yield behavior, stiffness degradation under cyclic loading, initial elastic stiffness Material Nonlinearity- One point plastic hinge, Numerical integration - Newmark Method (45). Time History - IS 1893 Spectrum compatible, 3 numbers, The average of the results obtained from the three time histories Imperial Valley, North Ridge and Lander with Time step of integration = 0.01,0.02, 0.02 second and Number of sample points = 4000, 1969, and 2200 respectively have been reported.

3. Design Computations

The analysis using FBD, CSM and NLTH was carried out using SAP 2000 and DDBD through computational algorithm. In all the cases the response spectrum given in code with 5% damping ratio was considered. The moment curvature relation for nonlinearity of hinge was derived from the section designer incorporated in (*SAP2000*) (46). The plastic hinge length was determined by the eqn (3). The Newmark time integration method was used for the nonlinear time history analysis.

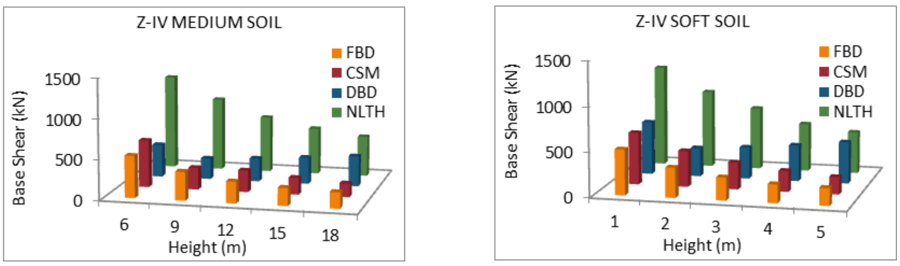
4. Discussion of Results

Base Shear is a very important parameter for the seismic design of bridges. Accordingly, the results obtained by various methods for different soil types and seismic zones have been presented and discussed. The base shear versus pier height for each zone for Medium and Soft Soils have been shown in Fig. 2, Fig.3 and Fig.4.



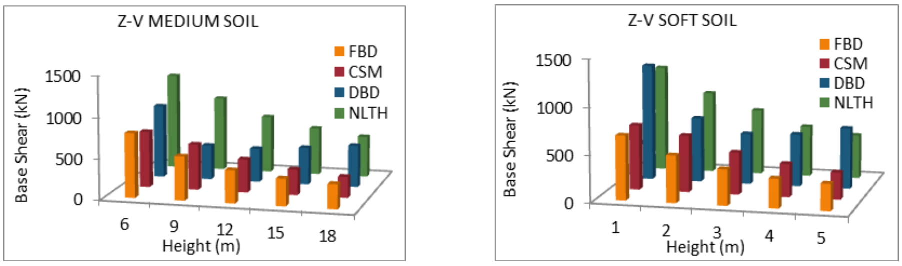
(a) (b)

Fig. 2 Base shear versus pier height for Z-III (a) Medium soil (b) Soft soil



(a) (b)

Fig. 3 Base shear versus pier height for Z-IV (a) Medium soil (b) Soft soil



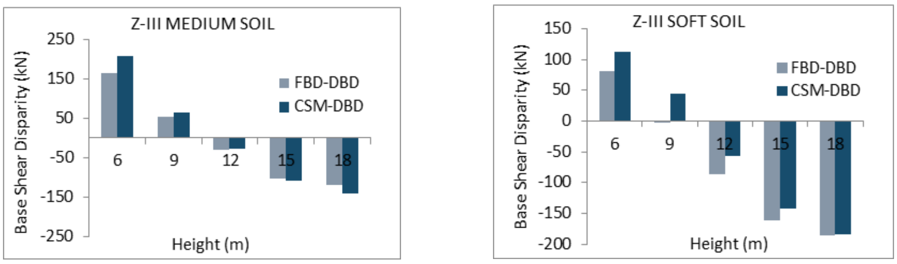
(a) (b)

Fig. 4 Base shear versus pier height for Z-V (a) Medium soil (b) Soft soil

Observations drawn from Fig. 2 to Fig. 4 are as under:

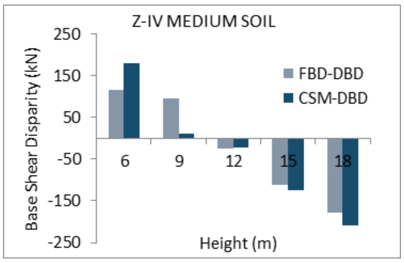
- The values of base shear obtained by different methods differ.
- In general, the values of base shear obtained by FBD, CSM and NLTH decrease with increase in pier height. The decrease in base shear is because of increasing flexibility.
- The values of base shear by NLTH are much higher than those by other methods except for pier height 6m in zone-V for soft soil.
- In case of DDBD, the values of base shear generally decrease from 6m to 9m or 12m pier height and then increase. The reason is that for the design displacement less than corner displacement calculated at equivalent damping, the time period increases with increasing height leading to decrease in member stiffness. Further, in case of design displacement more than corner displacement the effective time period remains constant while the design displacement is iterated to rectify the design ductility leading to decreased ductility. The decreased ductility leads to increased base shear. It is desirable that the design displacements should always be closer to the corner period displacement.
- The values of base shear obtained by CSM are slightly higher (maximum about 10%) in comparison of that by FBD. Both the method yield more or less same result, since both methods use the same response spectrum.
- The values of base shear obtained by FBD, CSM and DDBD were found to be less than that obtained from NLTH (except for one case) implying that the structures are under designed for economic considerations.

To study the variation of pattern of base shear disparity, the same were plotted against pier height. The base shear disparity between FBD and DDBD and between CSM and DDBD versus pier height for each seismic zone and various soil types are shown in Fig.5 and Fig. 6 and Fig. 7.

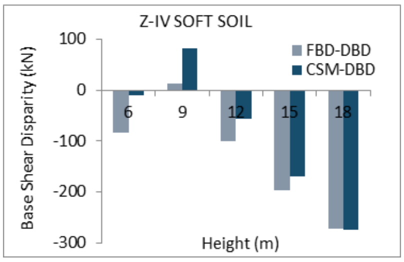


(a) (b)

Fig. 5 Base shear disparity versus pier height for Z-III (a) Medium soil (b) Soft soil

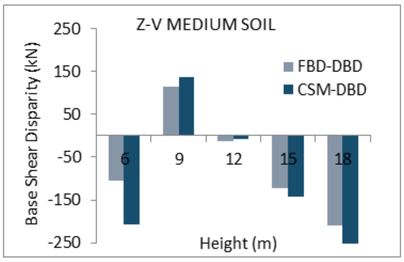


(a)

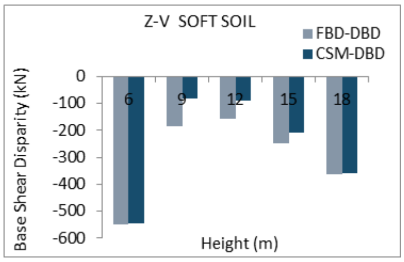


(b)

Fig. 6 Base shear disparity versus pier height for Z-IV (a) Medium soil (b) Soft soil



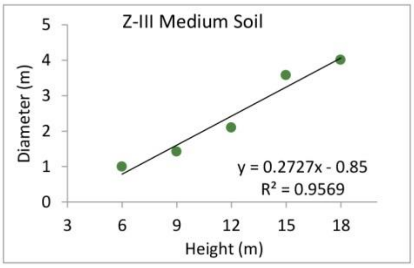
(a)



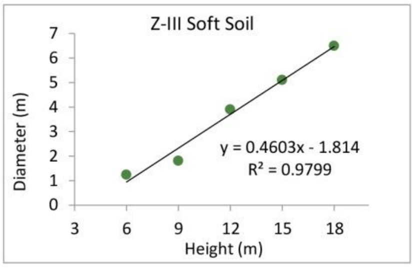
(b)

Fig. 7 Base shear disparity versus pier height for Z-V (a) Medium soil (b) Soft soil

The variation pattern shows that the difference in base shear is minimal in the range of Pier height 9m to 12m and this difference increases beyond this range on either side. This implies that for certain range of stiffness, the values of base shear by FBD, CSM and DDBD will be in close agreement. Since the values of base shear by FBD and CSM were quite close, studies were carried out to determine the diameter of the pier for which the base shear values were quite close for pier heights 6m, 9m, 12m, 15m and 18m for each seismic zone and different soil types. From these results, relationship between pier height and diameter were developed which are shown in Fig.8 and Fig.9 and Fig.10.

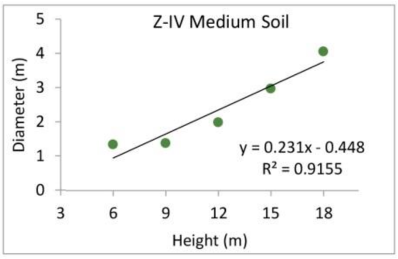


(a)

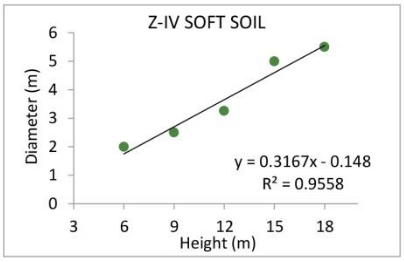


(b)

Fig. 8 Pier diameter versus pier height for medium soil (a) Z-III (b) Z-IV (c) Z-V

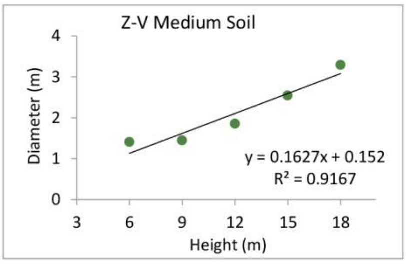


(a)

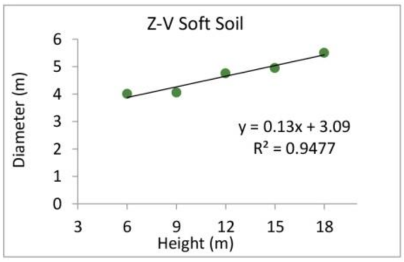


(b)

Fig. 9 Pier diameter versus pier height for medium soil (a) Z-III (b) Z-IV (c) Z-V



(a)



(b)

Fig. 10 Pier diameter versus pier height for soft soil (a) Z-III (b) Z-IV (c) Z-V

The pier diameter and height relationship as shown in Fig. 6 and Fig. 7 could be used to determine the diameter for chosen height for which the base shear values obtained by both FBD and DDBD methods would be of the same order and can be refined with adjustment in pier diameter. Further it is observed that the required diameter increases with increasing pier height and the values appear to be reasonable.

The ratios of pier height to diameter were computed for considered soil types and seismic zones. These ratios increase with increase in height for different soil types in each seismic zone. The average ratios were found to be 5.6 and 3.25 for medium and soft soils.

5. CONCLUSIONS

The capacity spectrum method and direct displacement design although give an insight to the behavior of the bridge pier through the various parameters such as yield displacement, ductility and equivalent viscous damping but these methods give different responses due to difference in approach and assumptions of the method. The DDBD approach sets the target displacement based on the ductility and drift limit whereas CSM define the displacement as the meeting point of the capacity of the section and the demand on the section. FBD calculated base shear based on the response reduction factor to account for inelastic structural behavior. The values obtained from CSM are closer to FBD and both CSM and FBD follows similar decreasing base shear trend variations with respect to increasing height, Contrarily, the values obtained from DDBD indicate considerable difference and shows decreasing and then increasing base shear trend with increasing height. NLTHM predicts the maximum base shear demand.

To achieve the design of the bridge pier of specified height in any seismic zone founded on any type of soil which could satisfy the code provisions as well as performance criteria (design base shear values being in close proximity by both FBD and DDBD method, it is suggested that an approximate value of diameter could be based on the ratio of height to diameter around 5.6 and 3.25 for medium and soft soil respectively, which could be refined through adjustments in pier diameter to satisfy both criteria. However, the practical requirement, especially in case of smaller pier heights, has to be kept in view. In this process the performance of the structure could become reliable during the earthquake motions.

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SEIZMIČKA ANALIZA ŠIPOVSKIH TEMELJA STUBOVA MOSTA METODAMA SILE I POMERANJA

Seizmička analiza stubova mosta oslonjenih na šipove zahteva posmatranje (kinematičke I inercijalne) interakcije tla-šipa-konstrukcije.Ovaj rad predstavlja projektne sile koje se stvaraju za stubove mostova sa promenljivom visinom i konstantnim presekom u srednjem i mekanom tlu u seizmičkim zonama i koristi razvijeni analitički pristup da uzme u obzir doprinos interakcije tle-šip-konstrukcija. Rezultati su pokazali da se razlika u maksimalnom smičućem naponu između sila dobijenih metodom sile i metodom pomeranja kao i ona između raspona nosivosti i metode pomeranja smanjuje kako se ovećava faktor vitkosti stuba. Maksimalni smičući napon dobijen nelinearnom analizom istorije ponašanja se pokazao mnogo većim nego kod drugih vrsta metoda. Odnos između visine i preseka stuba je razvijen za različite vrste tla i seizmičke zone tako da su maksimalni smičući naponi dobijeni metdom sile i metodom pomeranja istog reda. Ukupna vrednost faktora vitkosti je takva da bi lom stuba bio posmatran kao niski stub u srednjem i mekanom tlu.

Ključne reči: *Temelj stuba mosta, interakcija tla-stuba-konstrukcije, Projektovanje na osnovama sile, Projektovanje na osnovu direktnog pomeranja, Metod istorije nelinearnog vremena i Metod spektra nosivosti.*