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Effect of Cap Beam on Seismic Behavior of Contiguous Pile Quaywalls

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ABSTRACT

Cross section of contiguous pile type walls consists of discrete piles lying alongside of each other, however existence of relatively heavy cap beam which connect top of piles to each others, affect integrity of wall cross section. To study cap beam effect on these types of walls, numerical Modeling based on finite element method using software ABAQUS was performed. P-y method was used in order to model soil-pile interaction. Because seismic loading prevails in many design cases in Iran, in present study seismic behavior of the wharf was considered. A parametric study based on variation in cap beam dimensions was performed and comparison between equivalent 2D and 3D models was key factor to evaluate beam effect. Results showed that there is no major difference between results of 2D and 3D models and thus cap beam is capable of dividing soil pressures evenly between tie rods. Also, it was found that slight variation in cap beam dimensions has an insignificant effect on wharf behavior. So, it could be concluded that contiguous pile wharf can be modeled in 2 dimensions and expensive time consuming 3d modeling could be avoided.
KEYWORDS: capping, contiguous pile wall, p-y method, soil-pile interaction, ABAQUS.

1. INTRODUCTION

Nowadays, ship capacities are increasing and berthing structures with more strength and larger widths and draughts are required. One of most applicable type of wharves is sheet pile wall, but with increasing in wall height, and consequently increasing in section bending moment demand, flexible type sheet pile walls such as typical Z and U section walls cannot withstand bending stresses. So, one solution is use of piles functioning as wall cross section that provides enough bending modulus and can be driven deeper into soil layers. One type of pile walls is contiguous pile wall [1-2]. A contiguous pile wall consists of bored cast-in-place concrete piles along the line of the wall or steel driven piles. The dimension of the gap between the piles can be varied to suit site dimensions and the specific ground conditions within a typical range of 50–150 mm. large spacing is avoided as it can result in caving of soil through gaps. Diameter and spacing of the piles is decided based on soil type, ground water level and magnitude of design pressures. Verticality tolerances should be considered to ensure either that the potential gap between piles does not increase unacceptably with depth and that the piles do not overlap [1-2].

A major concern in designing of this type of walls is that the wall cross section is not continues and piles may have no connection to each other. One of the main structural elements of wharves is cap beam which assists equitable pressure distributions in piles [2]. Indeed, this beam provides facilities for berthing and mooring of ships but in contiguous pile type wharves, it can play another role that is waling. Wale in sheet pile walls is a beam that transfer soil pressure between wall and tie rods.

In this paper, the effect of cap beam on steel contiguous pile walls behavior under seismic loading was considered. For this purpose a contiguous type wharf was modeled in both 2 and 3 dimensions and cap beam dimensions are varying through five studied models. The two dimensional models are representative of plane strain behavior of the wall. Comparison between results of these two types of modeling was the key criterion for assessing beam effect on the wall discrete system. P-y method is used in order to model soil-pile interaction.

2. MATERIALS AND METHODS

2.1. Soil-Pile Interaction, P-Y Method

In this research p-y method is applied to model soil-pile interaction. By accepting Winkler's foundation assumption (1876) that each layer of soil responds independently to adjacent layers, a beam and discrete spring system may be adopted to model pile lateral loading. Although this assumption ignores the shear transfer between layers of soil, it has proven to be a popular and effective method for static and dynamic lateral pile response analyses. In this method, the soil-pile contact is discretized to a number of points where combinations of springs and dashpots represent the soil-pile stiffness and damping at each particular layer. These soil-pile springs may be linear elastic or nonlinear; p-y curves typically used to model nonlinear soil-pile stiffness have been empirically derived from field tests, and have the advantage of implicitly including pile installation effects on the surrounding soil, unlike other methods [3-5].

The Winkler method is a beam on elastic foundation method of analysis. In its purest form, the tieback wall is considered to be a continuous flexural member with stiffness EI that is supported by a set of infinitely closely spaced

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soil springs and discrete tieback anchor springs. The soil springs are preloaded to at-rest pressure conditions to represent the condition that exists prior to excavation. As excavation takes place (soil springs on the excavated side removed), the wallmoves toward the excavation. This movement is the result of the at-rest preload in the soil springs located on the backside of the wall. To keep the system in equilibrium, the soil springs on the excavation side of the wall must increase their loads beyond at-rest. At ground anchor support locations, the tieback is represented by a prestressed (preloaded) tieback anchor spring that also contributes to system equilibrium. The soil springs can be linear elastic or elastoplastic with an elastic stiffness, k. The Winkler analysis method (illustrated as fig. 1) can be used in a staged excavation analysis or as a final analysis where the completed structure is "wished" into place without consideration of system displacements that occurred during each stage of construction [3-5].

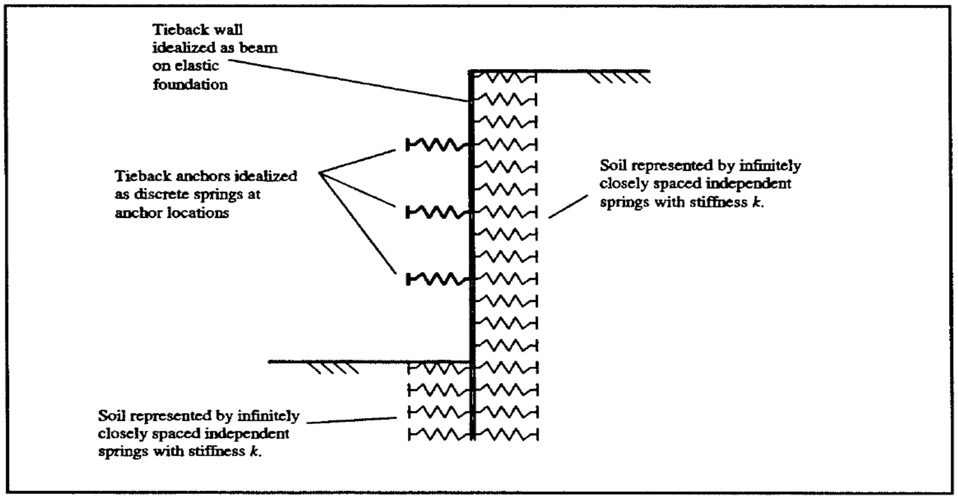


Fig. 1: Beam on elastic foundation method (Winkler)

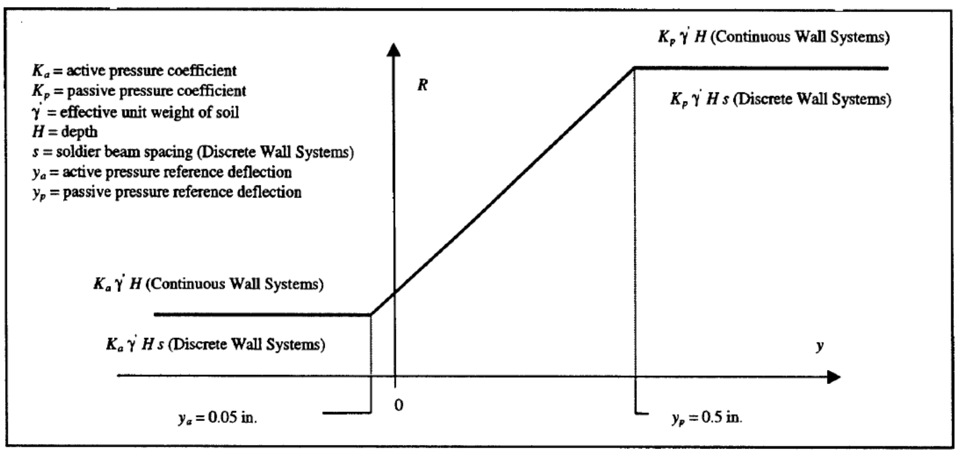


Fig. 2: Diagram illustrating p-y (R-y) curves for tieback walls in cohesion less soils

In wall-soil interaction, p-y curve is generally constructed by the Coefficient of Subgrade Reaction Method, the Reference Deflection Method, or the Poster Method. In the reference deflection method, an active reference deflection of 0.05 in. and a passive reference deflection of 0.5 in. are used to develop earth pressure-deflection relationships for cohesionless soils. The process is illustrated in

Fig. 2 for continuous wall systems and for discrete (soldier beam) systems [3-5].

2.2. General layout of the wharf

Contiguous pile wall modeled in this paper is a part of a real constructed harbor located in southern coastline of Iran. Main features of the wharf are summarized in Table 1.

Table 1: Main characteristics of the wharf

deck level	dredge level	pile end level	Tendon level	RWL
+5 mCD	-15.2 mCD	-21.55 mCD	+2.75 mCD	+0.57 mCD
CD= Chart Datum,	RWL= Residual Water Level			

2.3. Pile

Piles of the wharf are made of steel material with the dimensions of 1.42 m in diameter and 19.1 mm in thickness. Steel material characteristics are summarized in Table 2 .All piles are clamped to the cap beam and have no connection to each other's. In 2D model, piles are modeled by 1-D beam elements and in 3D models by 3-D shell elements.

Table 2: Steel piles material characteristics

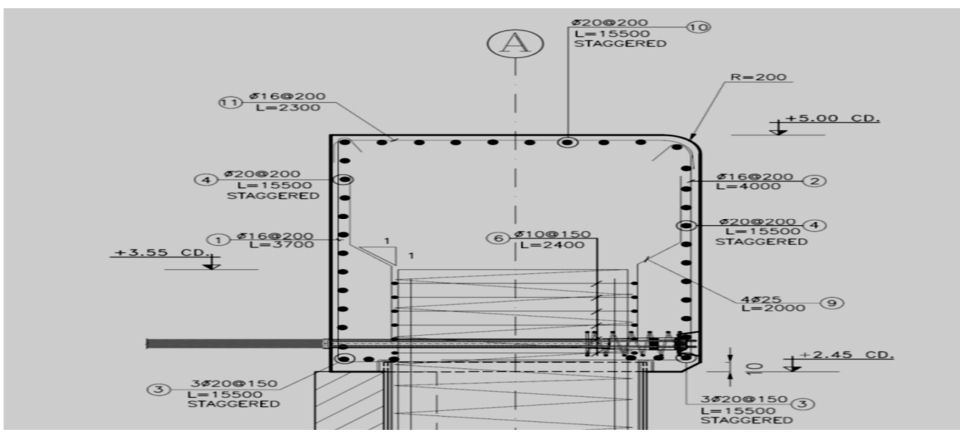
Yield strength ,fc (Mpa)	**30**
Modulus of elasticity, E (Gpa)	28.5
Yield strength of steel, fs (Mpa)	210
Density (kg/m3)	2500
Poison coefficient	0.2

2.4. Cap beam

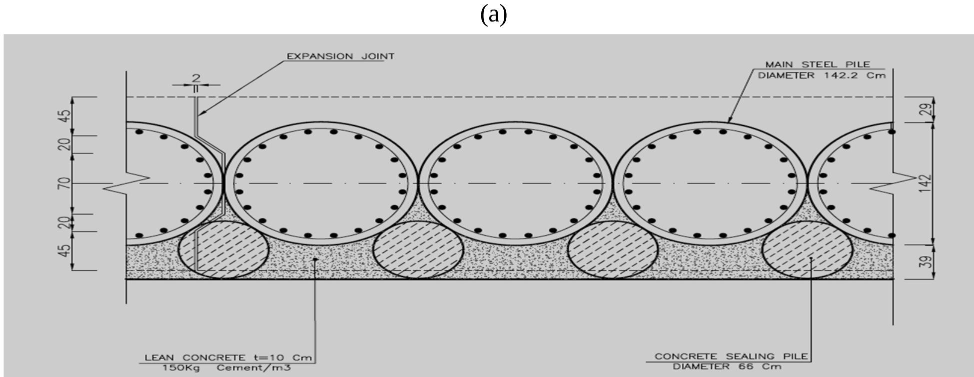
Cap beam in the model is reinforced concrete beam that has expansion joint at every 15.62 m. In 2D models, beam is modeled as 2-D plane strain element. In 3D models, beam is modeled as 3-D stress solid element. Steel rebar are modeled with beam elements and are embedded in concrete region. CDP model (Concrete Damage Plasticity) is used as constitutive model for concrete material [6]. The Concrete Damaged Plasticity Model uses the concept of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behavior of concrete. Reinforced concrete material characteristics are summarized in Table 3. Section of capping is shown in Fig. 3.

Table 3: Reinforced concrete material characteristics

Yield strength ,fy (Mpa)	**360**
Modulus of elasticity, E (Gpa)	210
Poison coefficient	0.3
Density (kg/m3)	7800



(a)



(b)

Fig. 3: Cap beam detail - (a): section (b): plan

2.5. Soil

The soil reaction to pile movement during transient seismic loading comprises stiffness and damping components [7]. In the present study, the soil stiffness is established using the p-y curve (lateral soil resistance versus lateral soil deflection) approach.

The soil pressure or anchor force exerted on the wall at any point is assumed to depend only on the displacement at that point (i.e., the Winkler assumption). In effect, the Winkler assumption results in treating the soil and anchors as isolated translation resisting elements.

Soil springs are generated using reference-deflection method (refer to Ref. [2]) that is suitable in sandy soils, because in sandy soil, with increasing in depth, soil stiffness increases and according to fixed reference deflection of springs along soil depth and increasing of soil ultimate strength with depth, this method agree with real behavior of sandy soils. Soil characteristics are summarized in Table 4.

Limiting forces at any points are calculated according to following equations [5].

a. For the curve below node *i* in Fig. 4.

(2.1) $$F_{ai} = \frac{h}{6}(2p_{ai} + p_{aj}) \quad , \quad F_{pi} = \frac{h}{6}(2p_{pi} + p_{pj})$$

b. For the curve above node *j* in Fig.4:

(2.2) $$F_{aj} = \frac{h}{6}(p_{ai} + 2p_{aj}) \quad , \quad F_{pj} = \frac{h}{6}(p_{pi} + 2p_{pj})$$

Soil limiting pressures should be calculated at two conditions, first at ordinary condition without imposing seismic effect, and second at earthquake condition. At latter condition, pressures are calculated from mononobe-okabe equations and bellow the water table apparent seismic coefficient is considered according to Ref. [8]. Seismic coefficient can be calculated using Eq. (4.3) according to Ref. [9].

(2.3) $$k_e = 0.6\frac{a_{\max}}{g}$$

The soil damping provides a major source of energy dissipation in pile-soil systems subjected to dynamic loading. In the present study, the damping component of the soil resistance is represented by a dashpot whose coefficient is established based on the Berger et al model [7, 10], i.e.:

(2.4)$C_L=4B\rho V_s$

Where B = pile diameter, Vs = soil shear wave velocity and ρ = soil unit density.

Table 4: Soil characteristic

type	phi	E (Mpa)	poisson	γ_d	γ_s	G	Vs
sandy gravel	40	80	0.25	18	21	32	132

2.6. Prestressed Anchor Springs[2-4]

All anchors are assumed to be attached to the right side of the wall and to extend away from the wall to the right. The characteristics of anchors are shown in Table 5.

A flexible anchor acts as a nonlinear concentrated spring in which the anchor force varies with anchor deformation along its line of action. Assuming 350 kN for anchor prestressed force, anchor p-y curveis obtained as shown in Fig. 5.

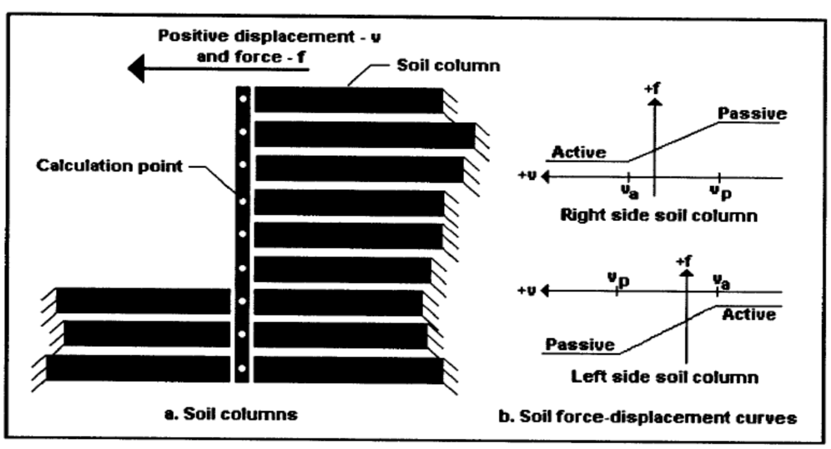


Fig. 4: Nonlinear soil springs

Table 5: Anchor characteristics

Type 6812			
D (in)	0.6	L (m)	38
n	12	E (Mpa)	195000
A (in2)	3.39292	k (kN/m)	11250

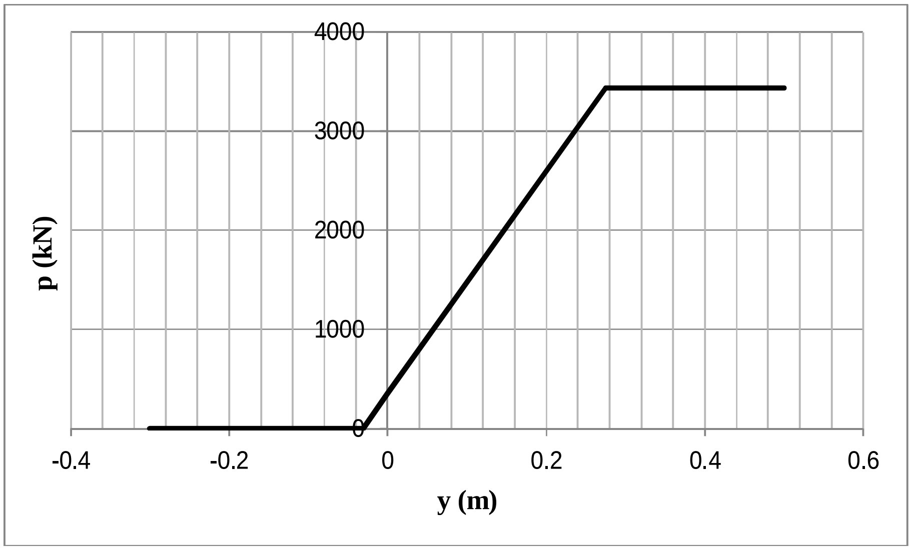


Fig. 5: Tendon p-y curve

2.7. Parametric study

For considering effect of cap beam on wall behavior, 5 models are constructed in both 2D and 3D according to variation of beam dimensions. Main models specifications are summarized in Table 6.

Table 6: Models capping specifications

model no	cap beam height (m)	cap beam width (m)
1-main	2.55	2.1
2	3.3	2.1
3	4.05	2.1
4	2.55	2.5
5	2.55	3

2.8. Analysis

Two step of analysis are defined to analyze the static/dynamic behavior. First analysis step is static in which static boundary condition is imposed to the model and ordinary condition for soil springs is assumed. After static equilibrium of the model, dynamic analysis is started. Bandarabbas earthquake record in (Fig. 6) with PGA of 0.15g is selected as the external seismic load on the wall. The computed ground motion at different levels within the soil is then applied to the nodal boundary supports representing the support motions. Seismic condition is assumed for generating soil springs.

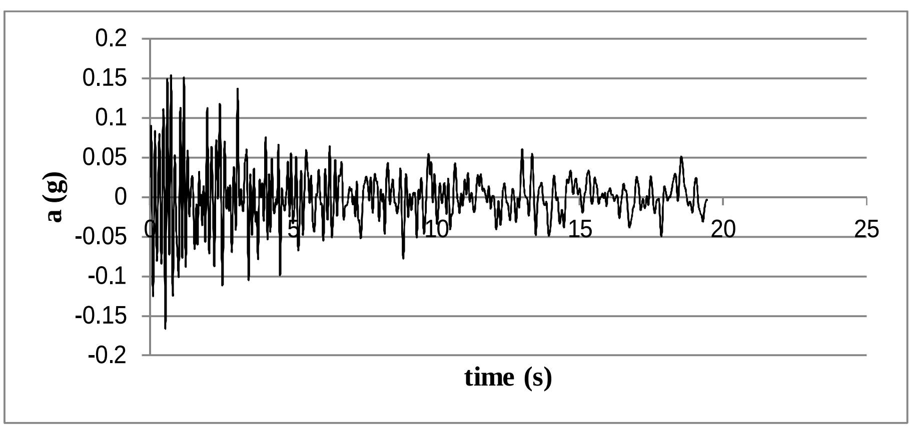


Fig. 6: Bandarabbas earthquake time history

3. RESULTS AND DISCUSSION

The history curve of wharf dynamic tip deflection in both 2D and corresponding 3D analysis for five models described before are depicted in following figures.

In model no. 1(Fig. 7), maximum deflection in 2D is 3 cm and that in 3D is 3.07 cm and difference is 2.28%. In no. 2 (Fig. 8), maximum deflection in 2D is 2.86 cm and that in 3D is 3.01 cm and difference is 5.14%.In no. 3(Fig. 9), maximum deflection in 2D is 2.91 cm and that in 3D is 3.01 cm and difference is 3.56%. In no. 4(Fig. 10), maximum deflection in 2D is 2.82 cm and that in 3D is 3 cm and difference is 6.37%.In no. 5(Fig. 11), maximum deflection in 2D is 2.98 cm and that in 3D is 3 cm and difference is 0.78%.

As it is shown in figures, it was found that global behavior of the wharf is not sensitive to beam parameters and only maximum values are changed. However, differences between 2D and 3D models are less than 7% that is negligible in practice.

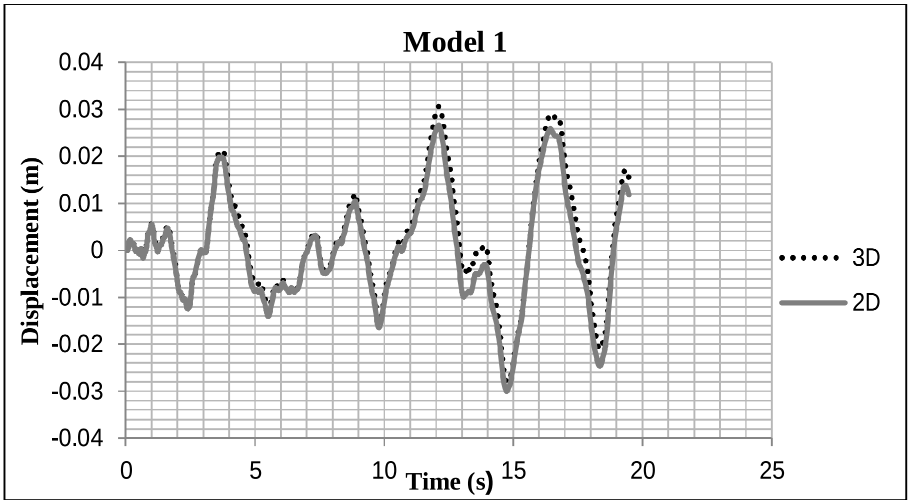


Fig. 7: Wharf dynamic tip deflection in model no. 1

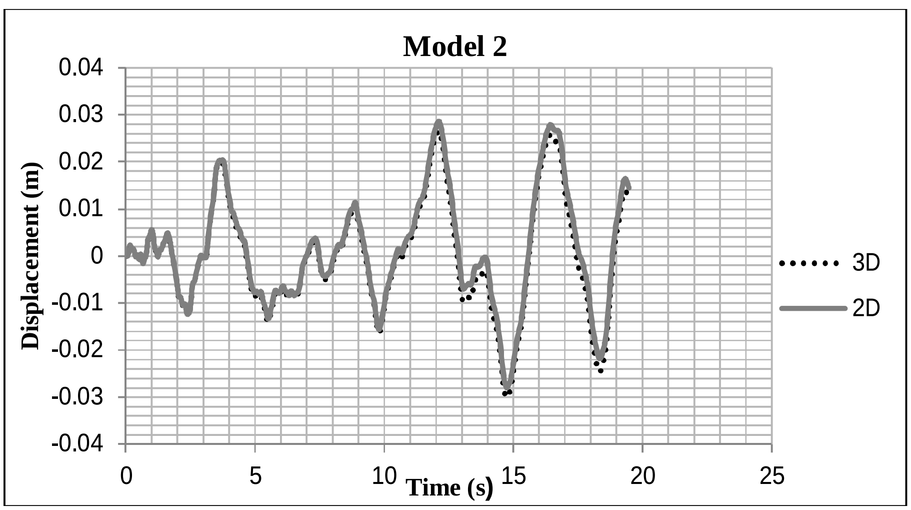


Fig. 8: Wharf dynamic tip deflection in model no. 2

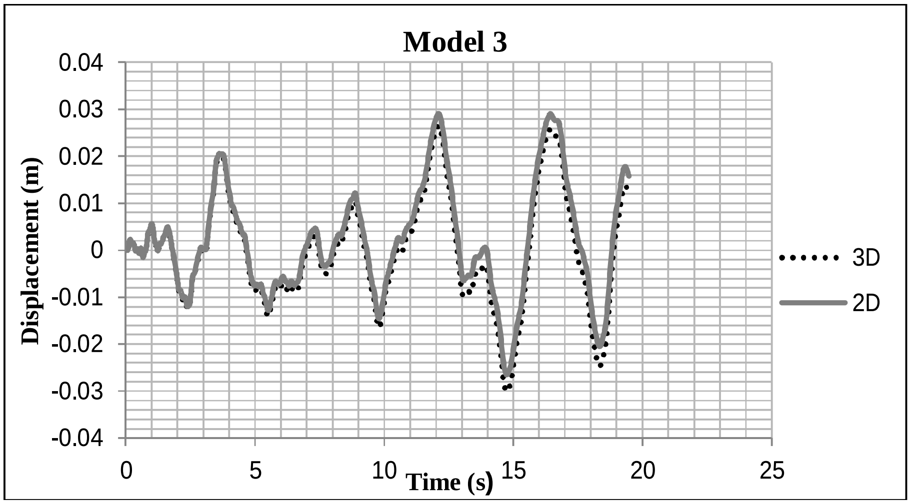


Fig. 9: Wharf dynamic tip deflection in model no. 3

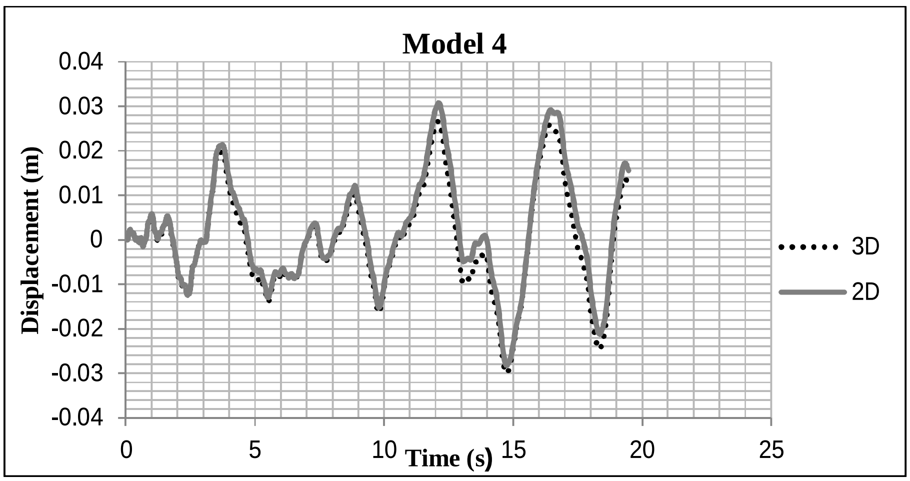


Fig. 10: Wharf dynamic tip deflection in model no. 4

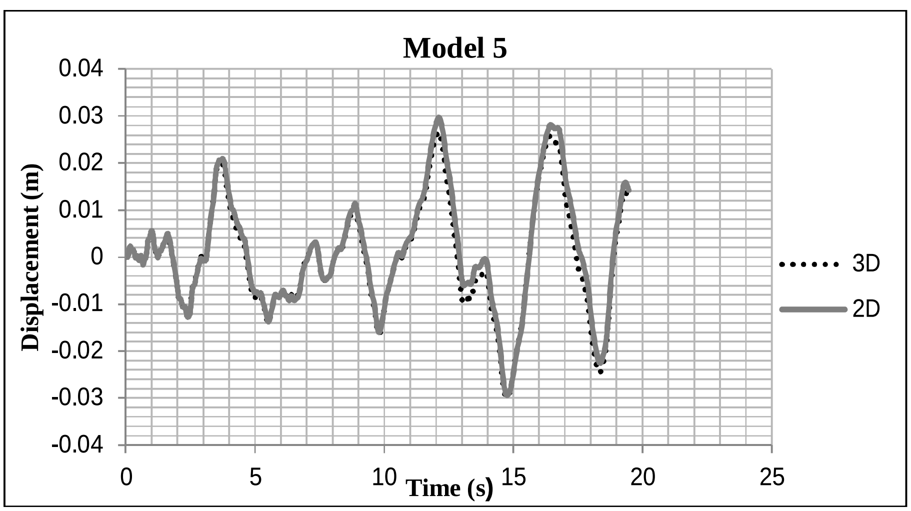


Fig. 11: Wharf dynamic tip deflection in model no. 5

In addition to comparison between 2D and 3D equivalent models, it is possible to compare between different 2D models respect to beam dimensions variations. Fig. 12 and Fig. 13 show this comparison. As it is shown there is no significant difference between models results. It should be noted that the wharf tip deflection is comprised of two main parts: First part is due to rigid deflection of the wharf to reach equilibrium in soil-structure interaction and second part is due to flexibility of wharf structural members. Thus, variation of deflection versus slight variation in structural parameters is not salient.

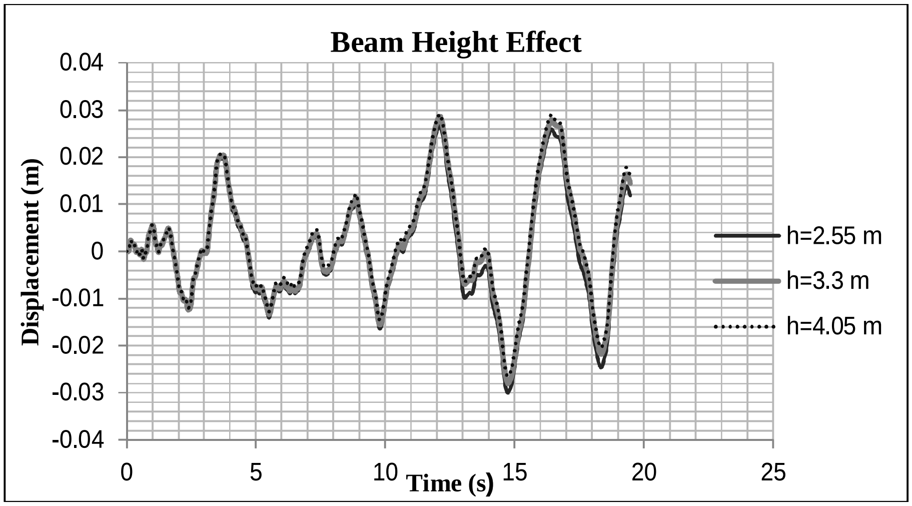


Fig. 12: Wharf dynamic tip deflection in different 2D models varying in beam height (h)

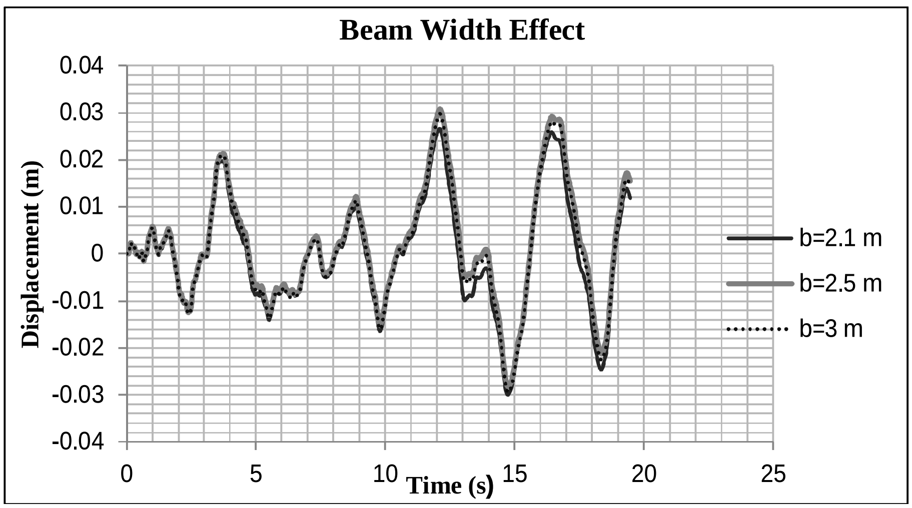


Fig. 13: Wharf dynamic tip deflection in different 2D models varying in beam width (b)

4. CONCLUSION AND RECOMMENDATIONS

Some useful results can be concluded from this study that summarized below:

- The contiguous pile in which cap beam functions as a wale, can be modeled in 2 dimension, so 3 dimensional analyses that is time and cost consuming is not necessary.
- The cap beam in contiguous pile walls can properly acts as a waling and there is no need to construct additional waling beam. In other words, it can transfer soil load from piles to tendon anchors and divide it almost equal between them
- Cap beam stiffness have a little effect on wharf seismic behavior and its tip deflection
- General seismic behavior of wharf is not sensitive to cap beam characteristics

For further studies recommended suggestions are mentioned following:

- The effect of soil parameters on behavior of these types of wharves can be considered. Constitutive models of soil behavior could be used instead of the p-y method.
- The effect of cap beam on behavior of other type of pile walls (such as contiguous pile wall consists of bored cast-in-place concrete piles) can be considered

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