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NON-LINEAR ANALYSIS OF AN INTEGRAL BRIDGE

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Abstract. This study describes the implementation of a 2-D finite element model of an integral abutment bridge (IAB) system which explicitly incorporates the nonlinear soil response. The superstructure members have been represented by means of three-node isoparametric beam elements with three degrees of freedom per node. The soil mass is idealized by eight node isoperimetric quadrilateral element at near field and five node isoparametric infinite element to simulate the far field behavior of the soil media. The non-linearity of the soil mass has been represented by using the Duncan and Chang hyperbolic model. The applicability of this model was demonstrated by analyzing a single span IAB. This study has shown that the soil nonlinearity has significant effect on the response of the structure, where the displacement that have been obtained on basis of nonlinear analysis is 1.5–2.0 times higher than that obtained from linear analysis. The stress magnitudes in the nonlinear analysis are also higher where in some point the difference reached almost 3 times.

Keywords: integral abutment bridge, soil structure interaction, nonlinear analysis, finite element analysis.

1. Introduction

The jointless bridge concept has become increasingly popular in recent years due to reduced maintenance costs associated with the expansion joints and abutment bearings, reduced corrosion and material degradation at the joints, and a better overall structural performance. Jointless bridges can be classified into four groups (Arsoy *et al.* 1999): Flexible arch bridges, Slip joint bridges, Abutment less bridges, and Integral bridges. Integral abutment bridges are the bridges which are generally built with their superstructures integral with the abutment and without expansion or contraction joints for the entire length of the superstructure, avoid expansion joints and movement bearings, that otherwise need regular maintenance.

The integral abutment bridge (IAB) is one of the significant developments in road bridge technology during the latter part of the 20th century. Unfortunately, the elimination of expansion joints, which can be a costly structural maintenance problem with conventional bridges, is the key benefit of an integral abutment bridge (IAB), and has resulted in other, unanticipated maintenance problems that turn out to be geotechnical in nature. In a sense, road agencies worldwide have simply exchanged one type of maintenance problem for another (Hovarth 2000). Integral abutment bridges have been used for road since the early 1930s in the U.S.A. However, they have seen more-extensive use worldwide in recent years because of their economy of construction in a wide range of conditions. Conducting the soil-structure interaction has always been challenging in analysis and design of integral bridges. Soil-structure interaction has a significant effect on the behavior of an integral bridge because; in integral abutment bridges the components of the superstructure are fully connected to the components of substructure.

Bridge deck are subjected to continual wear and heavy impact from repeated live loads as well as continual stages of movement from expansion and contraction caused by temperature changes, and/or creep and shrinkage, or long term movement effects such as settlement and soil pressure. Thus, this movement will be translated directly to the surrounding soil by the means of support (abutment and piles) in absence of expansion joints.

Research work on integral abutment bridge has started since the latter part of 20th century. From that time, many mathematical/ numerical models have been proposed to predict the real behavior of an integral bridge.

Springman *et al*. (1996) investigated the behavior of an integral abutment bridge under the cyclical temperature change on the bridge deck. Their study has shown that the cyclical temperature changes will result imposition of cyclical horizontal displacements to the backfill soil of the abutments.

Arsoy *et al*. (1999) modeled the IAB as a plane problem. The abutment was modeled using four node quadrilateral elements with linear stress-strain properties. The loads applied represent the forces exerted on the abutment by the superstructure. Finite element analyses has shown that the zone of surface deformation extends from the back of the abutment to a distance equal to about three to four times the height of the abutment. The movement of the abutment into the approach fill develops passive earth pressure that is displacement-dependent. The ground around the piles moves along with the movement of the abutment. The relative movement between

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the pile and ground is therefore reduced, resulting in relatively low shear forces at the top of the pile.

Non-Linear analysis of an integral bridge was carried by (Faraji *et al*. 2001). The presented work described the implementation of a full 3D finite-element model of an IAB system which explicitly incorporates the nonlinear soil response. A small parametric study was carried out on a sample bridge where the soil compaction levels in the cohesionless soils behind the wall and adjacent to the piles were varied. The abutment modeled as plate element, and the soil was represented by series of uncoupled Winkler spring. These results have shown that the level of compaction in the granular backfill strongly dominates the overall soil reaction, and that this reaction greatly impacts the overall structural response of the bridge system.

Noorzaei *et al*. (2004) used 3D modeling of abutment foundation backfill in integral bridge. In their study an attempt was made to carry out three dimensional finite elements modeling of integral abutment bridge- foundation backfill system subjected to temperature loading. Sixteen node isoperimetric brick element was used to model the abutment, foundation, backfill and supporting soil system. The finite element result has shown that, there is high stress concentration in the abutment and its neighborhood. The study concludes that, geotechnical study is essential for a realistic study.

Khodair and Hassiotis (2005) studied the effect of thermal loading on the soil/pile system using 3D, nonlinear finite element (FE) model. Material non-linearity is accounted for both, the piles and the soil. The displacements induced by temperature changes were measured and used as an input to the analytical model. The analytical results were compared with experimental data. The result has shown that, the influence of the lateral loads imposed by the superstructure on the piles is confined within a small volume of soil around the piles. As such, the lateral loading is not transferred to the MSE wall (Mechanically Stabilized Earth). However, this study was considering piles and the soil system only.

Fennema *et al*. (2005) determined the effect of the superstructure, thermal loading, and soil stiffness on the pile behavior. In this study, pile- soil media interaction was modeled using P-y curve. The result has shown that, the primary mode of movement of the integral abutment is through rotation about the base of the abutment, not longitudinal displacement of the abutment, as typically assumed for design.

Dreier (2008) investigated the importance of soil structure interaction to evaluate the allowable imposed displacement at the top of piers accounting for the cracking limit state. The results have shown that the influence of the stiffness of the foundation is very significant in the evaluation of Allowable imposed displacement. A low stiffness leads to a static system with a hinge at the base of the pier. On the contrary, a large stiffness leads to a static system clamped at the base.

The brief review of literature of physical and constitutive model of IAB bridge- foundations- soil system indicates that in most of the interactive analyses presented so far, the soil mass was modeled in the form of either a Winkler medium or linear analysis. The soil behavior is, as such, nonlinear. In this paper, an attempt has been made to take into account the nonlinear stress-strain response of the soil mass and its influence in the behavior of the structure as well as in the foundation. Furthermore, there is no evidence of work done on simulation of superstructure abutment piles and soil media as a single compatible unit.

The complex interaction between different components of the integral bridge with the foundation and surrounding soil requires more comprehensive modeling to reflect the actual behavior of bridge under different type of loading, and hence the numerical methods are often used for more advanced analysis (Juozapaitis *et al*. 2010), the objectives of the present study are:

i. To propose a 2-D numerical model using coupled finite and infinite elements.
ii. To account for soil non-linearity, using hyperbolic nonlinear elastic model.
iii. To apply the proposed physical and material models to an actual integral bridge.

2. Proposed physical model

In order to numerically simulate the integral bridge – foundations and soil media the following elements are utilized:

i. Three node isoparametric beam bending element with three degrees of freedom per node to represent the superstructure and pile. This beam element takes into account the effect of transverse shear forces and axial-flexural interaction (Hinton and Owen 1977; Godbole *et al*. 1990).
ii. Eight node conventional parabolic finite element to represent the abutment, and the soil mass. (Zienkiewicz *et al*. 2005).
iii. Five–node isoperimetric infinite element.
The main purpose of development of infinite elements is to model the unbounded domain. Infinite elements have been reported in Noorzaei *et al*. (1994) and Godbole *et al*. (1990).

A brief description of three node isoparametric beam bending element is discussed here in:

The proposed element is of isoparametric family with three degrees of freedom per node as shown in (Fig. 1a).

In general, the relationship between the strain components at any point and nodal displacements (Fig. 1b) is given by

$$\begin{Bmatrix} \varepsilon_x \\ \Phi \\ \dfrac{\partial\theta}{\partial x} \end{Bmatrix} = \begin{bmatrix} \dfrac{\partial N_i}{\partial x} & 0 & 0 \\ 0 & -\dfrac{\partial N_i}{\partial x} & N_i \\ 0 & 0 & \dfrac{\partial N_i}{\partial x} \end{bmatrix} \begin{Bmatrix} U_i \\ V_i \\ \theta_i \end{Bmatrix}, \tag{1}$$

where $\dfrac{\partial\theta}{\partial x}$ is a moment -curvature and Φ is the effective shear rotation.

The stress- strain relation may be expressed for the element as

$$\begin{Bmatrix} P_x \\ P_y \\ M_z \end{Bmatrix} = \begin{bmatrix} AE & 0 & 0 \\ 0 & S & 0 \\ 0 & 0 & EI \end{bmatrix} \begin{Bmatrix} \varepsilon_x \\ \Phi \\ \dfrac{\partial\theta}{\partial x} \end{Bmatrix}, \quad (2)$$

where A is the cross-sectional area of the beam, EI is the flexural rigidity, S is the shear rigidity ($S = A\ .\ G/\alpha$), and G is the shear modulus. P_x and P_y are the forces in the x and y directions, M_z is the bending moment and α is a factor to allow for warping stiffness-matrix.

The element global stiffness matrix in general is evaluated as

$$[K] = \int_{-1}^{1} [B]^T [D][B]|J|\,d\xi, \quad (3)$$

where [J] is the Jacobian determinant, and [K] is the global stiffness matrix.

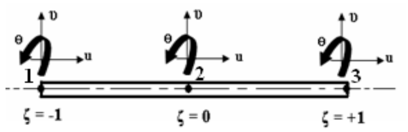


Fig. 1a. Parabolic isoperimetric beam element

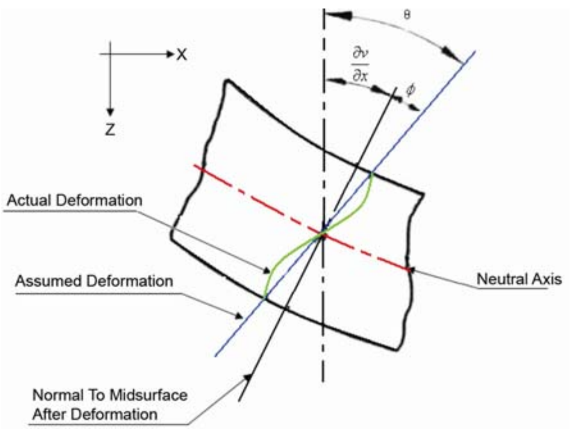


Fig. 1b. Cross sectional deformation of beam

The elements were used for the purpose of idealization of IAB- foundation- soil system are shown in Table 1.

3. Constitutive modeling

In this study, material non-linearity of the soil medium has been considered while the reinforced concrete is assumed to follow the linear stress-strain relationship. Moreover, a variation of the types of the soil with depth has also been considered to take into account four layers of the soil.

The hyperbolic model is attractive from the computational point of view. It is well suited for implementation in finite element programs and is applicable to virtually all types of soils and applied to analyze various types of soil-structure interaction problems. This model is useful for evaluating the movement in stable earth masses. The tangent modulus which is stress dependent is expressed as (Noorzaei 1991):

$$E_t = \left[1 - \frac{R_f(1-Sin\varphi)(\sigma_1-\sigma_3)}{2CCos\varphi + 2\sigma_3 Sin\varphi}\right]^2 KP_a\left(\frac{\sigma_3}{P_a}\right)^n, \quad (4)$$

where R_f = Failure ratio, P_a = Atmospheric pressure, n = exponent determining the rate of variation of E_t with σ_3, K = a modulus number, C = cohesion, φ = the soil friction angle, σ_1 = maximum principal stresses, σ_3 = minimum principal stress.

Table 1. Shape functions for elements were used in idealization of the structure

Type of element	Shape functions
8-node finite element	For corner nodes: $N_i = \frac{1}{4}(1+\xi\xi_i)(1+\eta\eta_i)(\xi\xi_i+\eta\eta_i-1)$ For midside nodes: $N_i = \frac{\xi_i^2}{2}(1+\xi\xi_i)(1+\eta^2)+\frac{\eta_i^2}{2}\cdot(1+\eta\eta_i)(1+\xi^2)$
5-node infinite element	$N_1 = -\xi(-\eta)\frac{(1-\eta)}{(1-\xi)}$ $N_2 = \frac{(1+\xi)(1-\eta)}{2(1-\xi)}$ $N_3 = \frac{(1+\xi)(1+\eta)}{2(1-\xi)}$ $N_4 = -\xi\eta\frac{(1+\eta)}{(1-\xi)}$ $N_5 = -2\xi\frac{(1-\eta\eta)}{(1-\xi)}$
3-node isoparametric beam bending element	$N_1 = -\frac{1}{2}\xi(1-\xi)$ $N_2 = (1-\xi^2)$ $N_3 = \frac{1}{2}\xi(1+\xi)$

4. Computer code

Based on the proposed physical and material models the finite element code which was developed by (Noorzaei *et al*. 1994), has been used in the present study. The current version of the program has one dimensional beam isoperimetric element with three degrees of freedom per node (u, υ, θ), two dimensional infinite elements and joint elements in its element library. The program can take into account the nonlinear stress-strain characteristics of soil. The nonlinearity can be handled by the incremental, iterative and a combination of incremental iterative technique.

5. Problem analyzed

The application of the proposed physical and constitutive model was shown by analyzing an actual integral bridge in Malaysia. Fig. 2 shows the front view of Sun GIA TITI GANTUNG Highway Bridge in Malaysia. The geometrical detail of the bridge are tabulated in Table 2.

Table 2. Geomtrical details of the bridge

Description	Length (m)
Total span length	39.40
Total width of bridge	11.50
Clear distance between parapets	10.50
Carriageway width	09.50

The bridge loading has been calculated based on loads for highway bridges (BD37/88, 1989). Since the materials nonlinearity is a major concern for this study, the load was calculated at the ultimate load state (*ULS*). The different loading combinations for the highway bridges are as follows:

i. First load Combination
HA loading consists of uniformly distributed load (*UDL*) and a Knife edge load (*KEL*)

$$1^{st}\, LC = (HA - UDL) + (HA + KEL)\,. \quad (5)$$

ii. Second Load Combination
HB loading is a load of abnormal vehicle. The *HB* vehicle replaces one lane of *HA* loading and is positioned for the worst effect.

$$2^{nd}\, LC = (HA - UDL) + (HB)\,. \quad (6)$$

Nominal loads shall be multiplied by the appropriate value of Y_{fl} to derive the load to be used in the analysis of bridge under the limit state.

$$1^{st}\, LC = 1.15DL + 1.75IDL + 1.5(HA(UDL) + HA(KEL))\,. \quad (7)$$

$$2^{nd}\, LC = 1.15DL + 1.75IDL + 1.3(HA(UDL) + HB(30unit))\,. \quad (8)$$

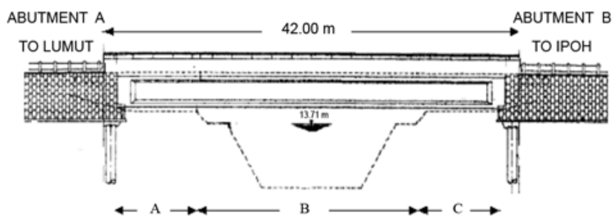


Fig. 2. Elevation View Schematic of Sun GIA TITI GANTUNG Malaysia

6. Finite-infinite element model

Finite element model of integral bridge-abutment pile-soil system are shown in (Fig. 4).

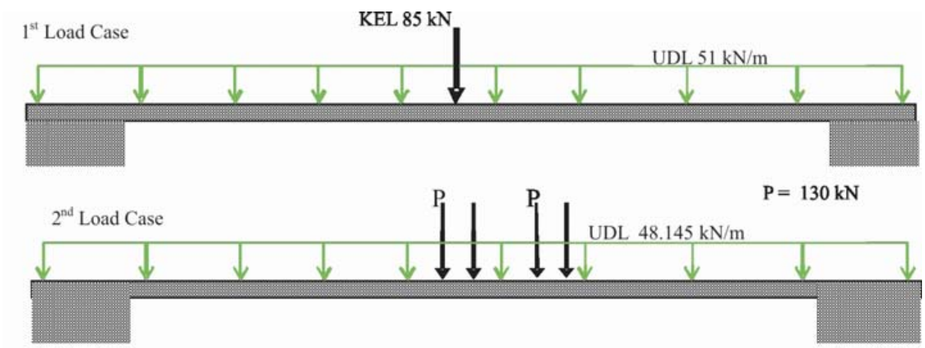
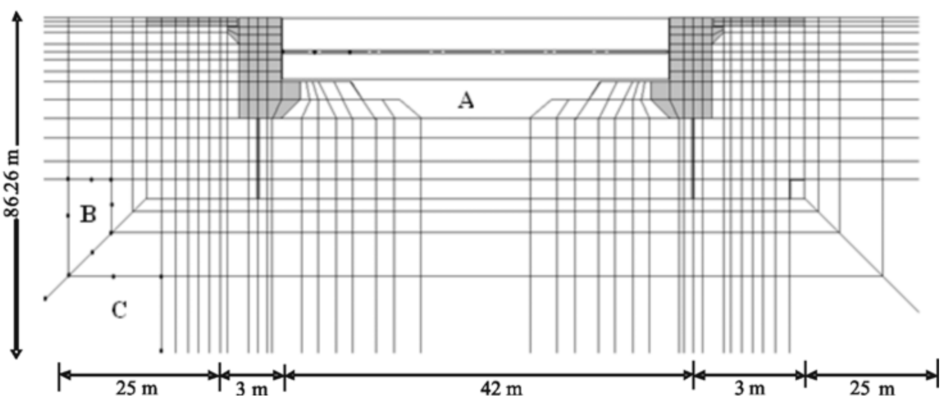


Fig. 3. Bridge loading



A is three-node isoperimetric beam element, B is eight-node isoperimetric element, and C is five-node isoperimetric infinite element

Fig. 4. Finite-infinite element discretization of IA bridge foundation-soil system

7. Evaluation of non-linear soil parameters

The parameters needed to define the tangent modulus of Elasticity E_t in equation (4) were obtained from the laboratory triaxial tests carried out on soil samples collected from the bridge site and presented in Table 3.

Table 3. Soil parameters

Type of the soil	Non-linear Soil Parameters				
	K	n	R_f	C (kN/m²)	φ (degree)
Clay	200	0.98	0.846	10	4
Sand Clay	200	0.995	0.88	22	19
Sand Slit	200	0.9	0.875	21	19
Dense Sand	198	0.82	0.855	70	22

8. Results and discussion

Figs. 5 show the deflection profile of girder for both, linear and non-linear analyses. The deflection profile of the superstructure logically follows the shape of a saucer, where the maximum deflection will be close to middle of the span and that depends on the position of load. The analysis result shows the profile deflection of slab for various combination loads, where (x = 0.5 L) middle span position was selected as the critical position. Obviously, the second load combination (*HB*) was found to be more critical as compared to the first load combination.

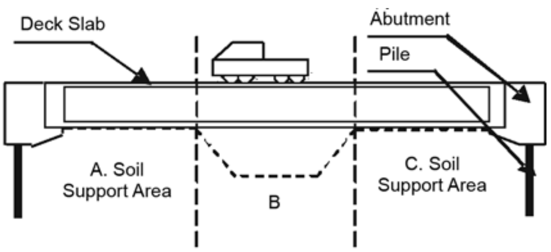


Fig. 5a. Diagram of the bridge

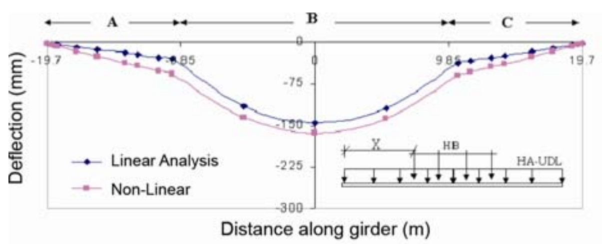


Fig. 5b. Deflection profile of girder for first load combination

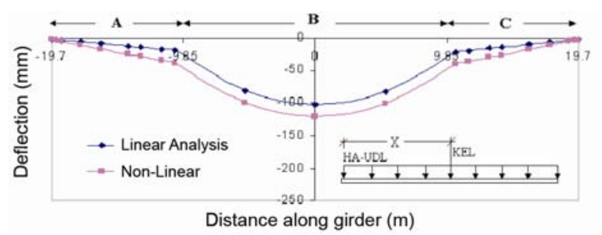


Fig. 5c. Deflection profile of girder for second load Combination

In the absence of the expansion joints, the deflection of the superstructure will be transferred to the abutment. The magnitude of this movement will be controlled by the type of load combination of load as shown in the (Figs. 6). The abutments will move forward at the top and backward at its bottom and is attributed to the deflection of the deck slab.

The finite element results have shown that the significance of soil nonlinearity in substructures behavior is more in comparison with the superstructures behavior. Thus, the soil is directly interacting with components of substructures. It is found that the displacement obtained using non-linear analysis is two times higher than that obtained using linear analysis. This is due to the fact that the tangent modulus, E_t is stress dependent.

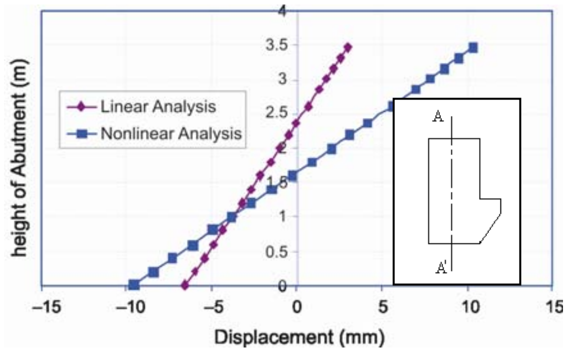


Fig. 6a. Lateral movement of abutment for first load combination a long A-A'

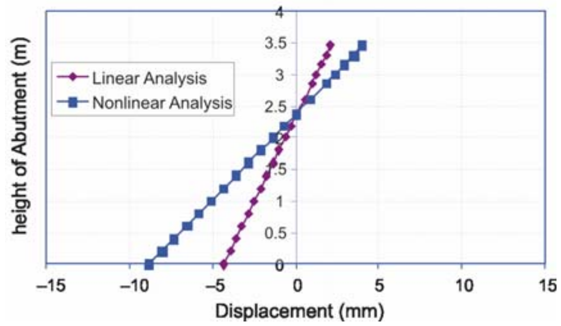


Fig. 6b. Lateral movement of abutment for second load combination a long A-A'

Lateral deflections of the piles below the abutment for both linear and nonlinear analyses are depicted in Fig. 7. The lateral displacement was not only due to the lateral loading, but also due to the vertical loading. Therefore, the vertical loading causes lateral displacement on planes perpendicular to the vertical axis of the pile along the pile length. It is usually of greatest interest to know the maximum displacement on these planes, where the vertical load carrying capacities of piles may be reduced due to lateral displacements.

The position of vertical load is very effective, the closes is load to the abutment, highs is the lateral displacement at the top of the pile. As load is applied to the abutment, it deflects and rotates as illustrated in Figs. 7. These movements generate reactions shear forces at the top of the piles which support the abutment.

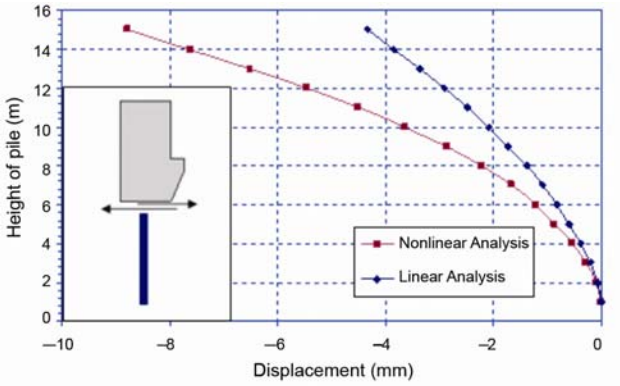


Fig. 7a. Lateral movement of piles for first load combination

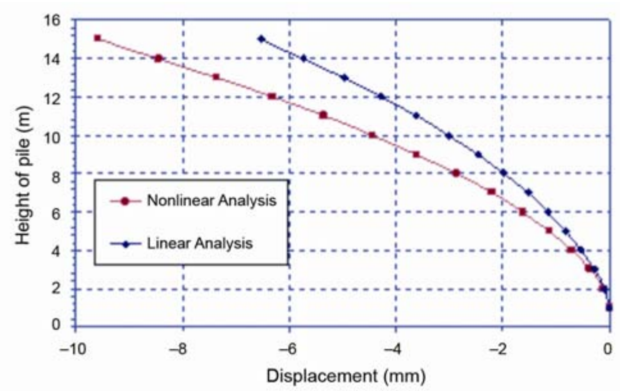


Fig. 7b. Lateral movement of piles for second load combination

Table 4 shows the lateral movement of pile. It is found that displacement obtained from non-linear analysis is 2–2.5 times higher than that obtained from linear anglysis.

Table 4. Lateral displacement of pile (mm)

Load Cases	Linear FEM	Non-Linear FEM
1st LC.	4.19	9.00
2nd LC.	6.58	9.8

In order to show the distribution of stresses within the abutment, Figs. 8 have been plotted to show the contour of concentration of σ_y of abutment for linear and nonlinear analysis.

The comparison between the results of linear and nonlinear analysis result has been shown in Fig. 9. It clearly observed from these plots that the affect of the nonlinearity is to redistribute the stresses compared to that obtained from linear analysis, where there is a reduction in the stresses at the center of the abutment. However, the stresses have been increased 1.5– 3 times at the concoction area between the abutment and approach slab and the pile as well.

The distribution of stresses along axis X-X, and Y-Y are presented in Figs. 10.

The finite element results show that the ground around the pile moves significantly as the load is applied to the abutment. In absence of expansion joint, the deflection of the superstructure due to vertical loads will move both of abutments forward and this movement will be

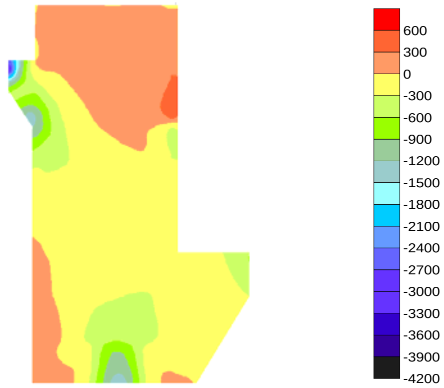


Fig. 8a. Contour of concentration σ_y of abutment for linear anglysis

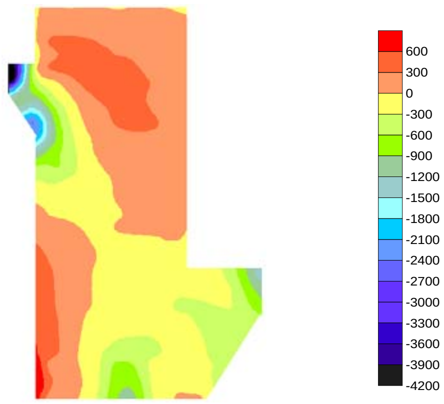


Fig. 8b. Contour of concentration σ_y of abutment for nonlinear anglysis

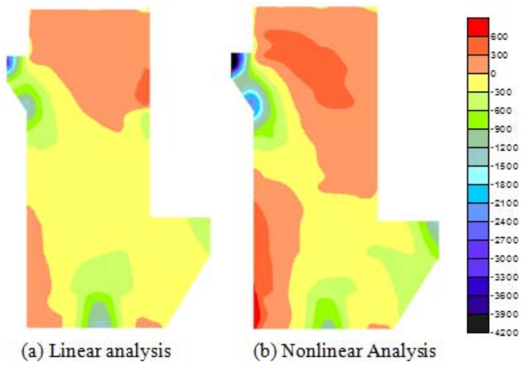


Fig. 9. Comparison of linear and nonlinear stresses

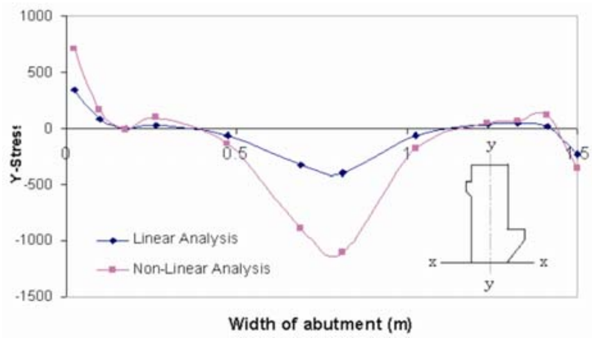


Fig. 10a. Y-Stress along X-X within the abutment

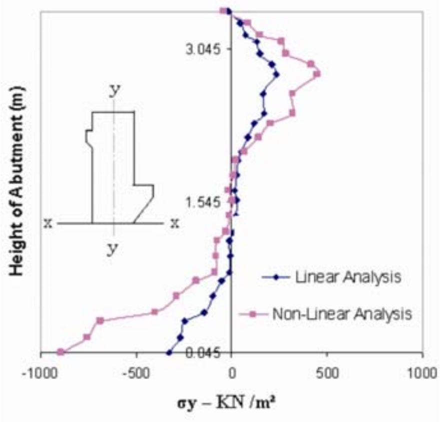


Fig. 10b. Y-Stress along Y-Y within the abutment

resisted by the backfill soil behind the abutment resulting in compressive stresses at the back of abutments and approach slab.

High concentration zone of compressive stress has been observed at the connection area of the abutment to the approach slab, which increased due to nonlinear effect.

On the other hand, tension zone has been observed at the low level of back of the abutment.

Fig. 11a shows the modeling of abutment, approach slab, and backfill soil.

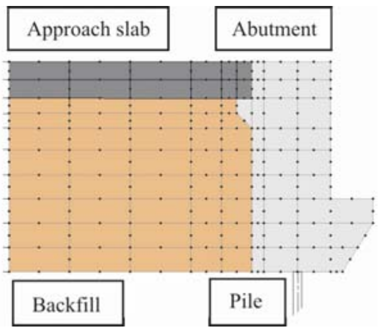
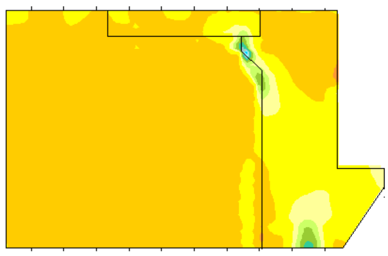
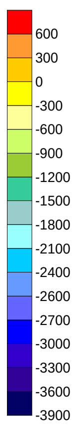
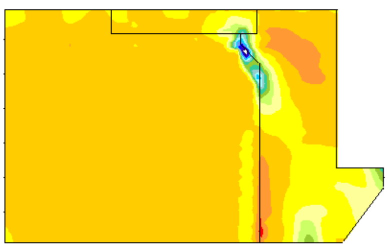


Fig. 11a. Shows the contour of concentration of σ_y in backfill for linear and nonlinear analyses



(a) Linear Anglysis





(b) Nonlinear Analysis

Fig. 11b. Contour of concentration of σ_y in backfill

9. Conclusions

One of the major uncertainty in the design of Integral abutment bridge (IAB) is the reaction of the soil behind the abutments and adjacent to the piles. The handling of soil-structure interaction in the analysis and design of integral abutment bridges has always been problematic. This study describes the implementation of a 2-D finite element model of IAB system which explicitly incorporates the nonlinear soil response. The non-linearity of the soil mass has been represented by using the Duncan and Chang approach, widely adopted for the hyperbolic model.

Based on the present study, the following conclusions can be drawn:

– The soil nonlinearity has significant effect on the results, where the displacements that have been obtained on the basis of nonlinear analysis are 1.5–2.0 times higher than that obtained from linear analysis.
– The stress concentration has been redistributed after the nonlinear analysis, where there is a reduction in the stresses at the center of the abutment and the stresses have been increased 1.5–3 times at the concoction area between the abutments and approach slab and the pile as well.
– The vertical load causes lateral displacement on planes perpendicular to the vertical axis of the pile along the pile length.
– The deflection of superstructure due to vertical loads will move both of abutments forward and this movement will be resisted by the backfill soil resulting in compressive stresses at the back of abutments and approach slab.

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NETIESINĖ INTEGRALINIO TILTO ANALIZĖ

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S a n t r a u k a

Straipsnyje aprašoma, kaip taikomas 2-D baigtinių elementų metodas tilto sistemai su integraliniais ramtais analizuoti, apimant ir netiesinę grunto elgseną. Antžeminės tilto dalies laikantieji elementai modeliuojami taikant trijų mazgų izoparametrinius strypinius elementus su trimis laisvės laipsniais kiekviename mazge. Grunto masyvui modeliuoti taikomi aštuonių mazgų izoparametriniai ketursieniai elementai arti tilto esančioje aplinkoje ir penkių mazgų izoparametriniai begaliniai elementai, imituojantys grunto terpės elgseną nuo tilto nutolusiose srityse. Grunto masyvo elgsenos netiesiškumas įvertinamas Duncan ir Chang hiperboliniu modeliu. Jo tinkamumas aiškinamas analizuojant vieno tarpatramio integralinį tiltą. Atlikti tyrimai parodė, kad grunto savybių netiesiškumas turi didelę įtaką tilto konstrukcijų elgsenai. Tilto poslinkiai, nustatyti taikant netiesinę analizę, yra 1,5–2,0 karto didesni už poslinkius, nustatytus taikant tiesinę analizę. Atlikus netiesinę analizę nustatyti įtempiai taip pat yra didesni, o kai kuriais atvejais skirtumas siekia beveik tris kartus.

Reikšminiai žodžiai: tiltas su integraliniais ramtais, grunto ir konstrukcijos sąveika, netiesinė analizė, baigtinių elementų analizė.

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