지반-구조물 상호작용을 고려한 기초모델링

Foundation Modeling Considering the Soil-Structure Interaction

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국문 요약 >> 지반-구조물의 상호작용은 구조물의 동적 해석 및 기초 설계에 있어 지대하 영향을 미침에도 불구하고 그 중요성이 간과되어 왔다. 이는 모델링 과정의 복잡성으로 인해 실무자를 위한 적절한 절차가 미비하다는 점에서 상당부분 그 이유를 찾을 수 있을 것이다. 본 연구에서는 먼저 구조물의 동적 해석이 필수적으로 요구되는 강진지역인 미국 캘리포니아에 위치한 Cal(IT)2 건물을 대상으로 지반 경계조건을 달리했을 시 해석상의 차이가 어느 정도 나는지를 검토해 보았다. 기초 모델링 기법의 하나인 Beam on Nonlinear Winkler Foundation Model을 Linear Matrix Inequalities Model Reduction 기법을 활용하여 보다 간략하게 사용할 수 있도록 하였다. 이렇게 하여 만들어진 대상 건물의 유한요소 모델과 실제 얻어진 가속도 데이터를 비교하여 제시된 방식을 통해 매우 우수한 해석 결과를 얻을 수 있음을 보였다.

주요어 지반-구조물 상호작용, BNWF 모델, LMI 모델리덕션

ABSTRACT >> Even with its significant influence on the dynamic analysis and foundation design of structures, sometimes the soil-structure interaction has been ignored during the design process. One of the reasons is due to the fact that the modeling procedures are too complicated to meet the requirements in practice. In this study, using the Cal(IT)2 building in California with high and frequent seismic activities, the analysis differences for different boundary conditions are reviewed. The Beam on Nonlinear Winkler Foundation Model, one of the foundation modeling methods, is modified for easy use by the Linear Matrix Inequalities Model Reduction Technique. The product of the proposed process is applied to create the Finite Element Model. The results show fairly good agreement with the real data acquired from the Cal(IT)2 building.

Key words Soil-structure interaction, Beam on Nonlinear Winkler Foundation Model, LMI model reduction

1. Introduction

When the response of the soil influences the motion of the structure and the response of the structure influences the motion of the soil, it is referred to as soil-structure interaction. For highly flexible structural systems, foundation displacements and rotations may be inconsequential small relative to those in the building and can be

safely neglected. However, for stiff structural systems, foundation deformations can represent a significant component of system flexibility. Ignoring foundation deformation in such cases results in mischaracterization of dynamic properties such as the fundamental mode frequency and the damping ratio.^{(1),(2)} As a result, it biases the engineering characterization of seismic demand.

There have been many efforts to make engineering guidelines for simple characterization of soil-structure-interaction (SSI) effects. One of the representative guidelines is intended for use with force-based characterization of seismic design, as is commonly used for new building construction. These procedures were introduced by ATC⁽³⁾ and an updated version is published in the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*.⁽⁴⁾ The ATC and NEHRP

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procedures modify the fixed-base building period and damping ratio for the effects of foundation compliance. The modified vibration properties (Termed “flexible-base”) are used with the site design response spectrum to obtain the base shear for seismic design. The foundation-soil characterization inherent to these procedures consists of foundation springs for translational and rotational deformation modes that depend on a strain-compatible soil shear modulus. The soil behavior is modeled as visco-elastic with no limiting shear stress.

The other guideline is intended for use with nonlinear static methods for structural performance assessment, as commonly used for building retrofit design.^{(5),(6)} In this approach, the structural performance is characterized by a nonlinear static pushover curve. The shape of the pushover curve, as well as the distribution of member shears and moments, can be sensitive to foundation modeling. Accordingly, the aforementioned documents provide recommendations for modeling the foundation-soil system as elastic-perfectly-plastic elements positioned at each footing. The elastic portion is based on the real part of well-known impedance functions for foundation lateral translation, vertical translation, and rocking. The plastic portion of the foundation springs is based on limiting soil pressures associated with bearing capacity failure(In the vertical direction) and sliding/passive failure(In the lateral direction).

Those guidelines are typically based in large part on representing the soil-foundation interaction in terms of elastic impedance functions that describe stiffness and damping characteristics. Such approaches are not able to capture the nonlinear behavior at the foundation level, which may involve formation of a temporary gap between the footing and soil, the settlement of the foundation, sliding, or energy dissipation from the hysteretic effects. A new research⁽⁷⁾ proposed Beam-on-Nonlinear-Winkler-Foundation (BNWF) model to overcome those shortcomings. The BNWF model consists of a system of closely spaced independent nonlinear inelastic springs that are capable of capturing gapping and radiation damping. Vertical springs distributed along the base of the sides of the footing capture the resistance to sliding.

Additional researches are required to provide further insight into nonlinear foundation-soil interaction and the benefits of performing the type of simulations enabled by BNWF model. The first issue to be solved is that this sophisticated model needs too much work to apply to practice. Currently, BNWF is available for 2-D analysis and the process still requires quite a work although simpler than previous suggested methods. Moreover, when it's developed to the 3-dimensional analysis, more complicated process is expected. It may raise a cost problem in design process. Second, the performance of this model against full-scale building has not been verified, especially under strongly shaken condition. Therefore, this model needs to be checked if it is applicable to make a global building model considering soil-structure interaction for system identification purpose.

The primary object of this research is to evaluate the effects of soil-structure interaction on identified modal frequencies. To achieve the goal, a high-fidelity and low order BNWF model is developed. The feasibility of the model is examined using measured data from the Cal(IT)2 building (See Figure 1).

In Cal(IT)2 building and in its foundation soil, 43 force-balanced accelerometers were installed during the

(a) Cal(IT)2 Building

(b) Sensor Layout

〈Fig. 1〉 Cal(IT)2 Building in UC Irvine Campus

construction. Each floor is instrumented with at least five sensors to capture the longitudinal, transverse, and torsional vibration modes. Sensors in vertical direction were mounted on shear walls at the ground level and the roof level to monitor the rocking motion of the structure. What makes this building unique is that 12 sensors were installed in the free-field and deep onto the bedrock, in addition to those on the building structure. This makes it possible to better understand the whole soil-structure system. The down-hole sensors are 3 m deep and the free-field sensors are approximately 10 m away from the structure.

2. Soil Condition at Cal(IT)2 Building Site

During the earthquake ground motions, soil starts to move and as a result, the modal frequencies of the soil-structure system decreases. This is due to the change in the shear modulus of the foundation soil during the earthquake, which can be examined by the strain history of the foundation soil. The displacement time history is obtained by double integration of the foundation and the down-hole acceleration measurements. The soil strain history is determined by dividing the relative displacement with the depth of the soil layer. Figure 2 shows the horizontal strain history of the foundation soil during Yucaipa Earthquake in 2005. It informs that the decrease in the shear modulus of the soil during the event is in the order of 20% compared to its original value. The soil shear velocity v_s is defined as

$$v_s = \sqrt{\frac{G}{\rho}} \quad (1)$$

where G is shear modulus, and ρ is dry density of soil.

From geotechnical report, it was found that the soil beneath the foundation is medium clay with shear wave velocity of 1269.7 ft/sec(=387.0 m/sec). The average dry density of the soil is 112.6 lb/ft³(=1803.7 kg/m³). Therefore, the initial shear modulus of soil is

$$G = \rho \cdot v_s^2 = \frac{112.6 \text{ lb} / \text{ft}^3}{32.2 \text{ ft} / \text{sec}^2} (1269.7 \text{ ft} / \text{sec})^2 = 5585.4 \text{ kips} / \text{ft}^2 = 267.4 \text{ MPa} \quad (2)$$

Following ASCE-41 and FEMA-356 (Table 1), similar result can be obtained. At the site where Cal(IT)2 building is located (Latitude: 33.686887, longitude: -117.825348),

(a) Soil Strain

(b) Shear Modulus Change

⟨Fig. 2⟩ Soil strain and Shear Modulus Curves

⟨Table 1⟩ Effective Shear Modulus Ratio (G/G_0)

Site Class	Effective Peak Acceleration, $S_{xs}/2.5$			
	$S_{xs}=0$	$S_{xs}=0.1$	$S_{xs}=0.4$	$S_{xs}=0.8$
A	1.00	1.00	1.00	1.00
B	1.00	1.00	0.95	0.90
C	1.00	0.95	0.75	0.60
D	1.00	0.90	0.50	0.10
E	1.00	0.60	0.05	-
F	-	-	-	-

〈Table 2〉 Dimensions of Cal(IT)2 Building Foundations and Bearing Strength (q_{ult})

Footing	Size	Thickness	q_{ult}	Footing	Size	Thickness	q_{ult}
F3	3'×3'	1'-0"	228	F8	8'×8'	2'-0"	1752
F4	4'×4'	1'-6"	416	F9	9'×9'	2'-6"	2270
F5	5'×5'	1'-6"	656	F10	10'×10'	2'-6"	2827
F6	6'×6'	1'-6"	953	F1487	14'-8"×7'	2'-9"	2630
F7	7'×7'	2'-0"	1330				

〈Table 3〉 Dynamic Stiffness of Cal(IT)2 Building Foundations in vertical Direction

	Surface (kips/ft)	Emb. factor	Embedded (kips/ft)
F3	56,771	1.51	85,952
F4	75,695	1.50	113,829
F5	94,618	1.41	133,743
F6	113,542	1.35	153,617
F7	132,466	1.37	181,595
F8	151,389	1.33	201,514
F9	170,313	1.35	229,467
F10	189,237	1.32	249,412
F1487	197,402	1.35	266,202

the effective shear modulus is 79% of the maximum shear modulus. By Terzaghi-Meyerhof equation, the ultimate bearing strength of soil is calculated (Table 2).

$$q_{ult} = \frac{1}{2} \gamma B N_{\gamma} + c N_c + D_f N_q \quad (3)$$

The capacity factors N_{γ} , N_c , N_q are varying according to soil properties and well known from many experimental studies. B is width of footing and D_f is depth from the surface to the bottom of the footing. c is cohesion of soil, and γ is specific weight of soil. Shape factors are needed for correction of N_c and $N_{\gamma} N_q$.

3. Foundation Modeling

3.1 Dynamic Parameters

A key step in current methods of dynamic analysis of SSI under seismic loading is to estimate the dynamic impedance functions associated with a rigid but massless function. However, application of computational methods to a specific engineering problem requires substantial expertise in idealizing the actual system, and entails significant data-preparation and computation expenses. As a practical alternative, several researchers have developed

〈Fig. 3〉 Direction & Dimension Notations of Footing

simplified methods, among them, Gazetas equations for shallow stiffness⁽⁸⁾ are widely used because it adequately reflects following key characteristics of the foundation-soil system and excitation: (1) the shape of the foundation-soil interface; (2) the amount of embedment; (3) the nature of the soil profile; and (4) the mode of vibration and the frequencies of excitation. Following the formulas and charts, the resultant dynamic stiffness and dashpot coefficients of Cal(IT)2 building foundations are calculated. Direction and dimension notations are shown in Figure 3. Here, as an example, the vertical stiffness of foundations is provided (Table 3).

3.2 Evaluation of Soil-Structure Interaction

As illustrated in Figure 4, the objective of system identification analyses is to evaluate the unknown properties

of a system using a known input into, and output from, that system. For analyses of seismic response, the system has an unknown flexibility that generates a known difference between pairs of input and output strong motion recordings. As indicated in Figure 5, parameters describing the fixed-base system are evaluated from input/output pairs that differ only the structural deformation U . Likewise, parameters describing the flexible base system are evaluated from strong motion pairs whose difference results from foundation flexibility in translation U_f and rocking θ , as well as structural flexibility. A comparison of fixed-base and flexible-base modal parameters provides a direct quantification of SSI effects.

In Cal(IT)2 building, in total 43 force-balanced accelerometers were installed, but not for measurement of θ . Introducing vertical measurements at each side of the building U_4 and U_5 , $\theta = (U_4 - U_5) / (\text{Width of building})$.

Three cases of base fixity are of interest in analyses of SSI: (1) Fixed-base representing only the flexibility of the structure; (2) Flexible-base representing the combined

flexibility of the complete soil-structure system; and (3) Pseudoflexible-base representing flexibility in the structure and rocking in the foundation.⁽²⁾ Using discrete time filter(DTF), transfer functions were obtained as Figure 6 for Yucaipa Earthquake.⁽⁹⁾

It was observed that under earthquake excitations, the identified modal frequencies increase as the modeling of the building base changes from the flexible base to the fixed base (Table 4). This implies that the fixed base model only reflects the structural response, while the flexible base model involves not only the structural response, but also the swaying and rocking motions. It also implies that if the soil properties are permanently changed after an earthquake, the modal values will not be same even under the same earthquake.

(a) Longitudinal Direction

(b) Transverse Direction

Figure 6: Transfer Function for Yucaipa Earthquake

Table 4: Identified Modal Parameters

	Yucaipa EQ- in Longitudinal Direction (Hz)	Yucaipa EQ- in Transverse Direction (Hz)
Flexible Base	2.76	2.76
Pseudo-flexible Base	2.80(+1.4%)	2.84(+2.9%)
Fixed Base	2.85(+3.3%)	3.04(+10.1%)

3.3 Beam on Nonlinear Winkler Foundation (BNWF) Model

This model is constructed with elastic beam-column elements to capture the structural footing behavior and zero-length soil elements to model the soil-footing behavior. As illustrated in Figure 7, inelastic material models such as QzSimple 1, PySimple 1, and TzSimple 1 in OpenSees were implemented to simulate vertical load-displacement behavior, horizontal passive load-displacement behavior against the side of a footing, and horizontal shear-sliding behavior at the base of a footing, respectively.

These three springs follow a nonlinear backbone resembling a bilinear behavior. PySimple 1 and QzSimple 1 have the capability to account for soil-foundation separation via gap elements added in series with the elastic and plastic components. The elastic material captures the far-field behavior, while the plastic component captures the near-field permanent displacement. As an example, in the elastic range, the QzSimple 1 is expressed as

$$q = k_{in}z \quad (4)$$

where q is load, k_{in} is initial elastic(Tangent) stiffness,

〈Fig. 7〉 Example of BNWF Model

and z is the displacement.

The elastic region(yield point, q_0) is defined by relating the ultimate load(q_{ult}) with a parameter C_r , i.e.,

$$q_0 = C_r q_{ult} \quad (5)$$

In the nonlinear portion, the backbone curve is described by

$$q = q_{ult} - (q_{ult} - q_0) \left(\frac{cz_{50}}{cz_{50} + |z - z_0^p|} \right)^n \quad (6)$$

where z_0 is displacement at the yield point and z_{50} is displacement at which 50% of the ultimate load is mobilized, which is defined as

$$z_{50} = \frac{0.52 q_{ult}}{k_{in}} \quad (7)$$

c and n are constitutive parameters controlling the shape of the post-yield portion. According to Matlock⁽¹⁰⁾ and API⁽¹¹⁾, $c=10$, $n=5$, $C_r=0.35$ for soft clay, and $c=0.5$, $n=2$, $C_r=0.2$ for drained sand. The ultimate bearing capacity (q_{ult}) can be obtained by the Terzaghi theory⁽¹²⁾. See Table 5 for more values.

As an example, BNWF model for F10 footing in Cal(IT)2 building is considered. For simplicity, only vertical elastic and inelastic springs in vertical direction are designed for the applied moment in x-direction(M_x).

- 1) Stiffness intensity ratio, $R_k = K_{end}/K_{mid}$ is 5, in this case.⁽¹³⁾
- 2) End length ratio, $R_e = L_{end}/L_s$, is 0.16 per ATC-40, therefore, $L_{end}=1.6 \text{ ft}$.

〈Table 5〉 parameters for nonlinear spring

Material Type	Soil Type	Recommended			References	Value used in this study		
		C_r	n	c		C_r	n	c
Qzsimple 1	clay	0.2	1.2	0.35	Reese & O'Neill (1988)	0.22	1.2	0.5
	sand	0.3	5.5	12.3	Vijayvergiya (1977)	0.36	5.5	9.29
Pysimple 1	clay	0.35	5	10	Matlock (1970)	0.35	5	10
	sand	0.2	2	0.5	API (1993)	0.33	2	1.1
Tzsimple 1	clay	0.5	1.5	0.5	Reese & O'Neill (1988)	0.5	1.5	0.5
	sand	0.5	0.85	0.6	Mosher(1984)	0.48	0.85	0.26

- 3) Spring spacing is 4.8 inch because minimum 25 springs are required along the footing length.

This model requires 25 plastic springs and the same number of elastic springs for one-strap model(2-D analysis). When it comes to the 3-dimensional analysis, 625 springs are needed. Moreover, a number of passive springs and sliding springs should be added considering horizontal loads. It is quite a number for just one footing. This issue raises a need of simplified method. A schematic plan and section for vertical spring design of F10 footing is illustrated in Figure 8. Though it's not shown here for simplicity, a number of lateral and frictional springs should be added in real situation.

3.4 State-Space Representation and Linearization

Rearranging Equation 6, the nonlinear tangent stiffness k_p which describe the load-displacement relation within the nonlinear region of the backbone curve may be expressed as

$$k_p = n(q_{ult} - q_0) \left[\frac{(cz_{50})^n}{(cz_{50} - z_0 + z)^{n+1}} \right] \quad (8)$$

As a test bed, 2-dimensional F3 footing receiving vertical load and moment is considered(Figure 9). For simplicity purpose, damping is not considered here.

$$k_{end} = \frac{0.02894}{(z + 0.000113)^{2.2}} \quad \text{at end effect area} \quad (9)$$

$$k_{mid} = \frac{0.005788}{(z + 0.000113)^{2.2}} \quad \text{at the other area} \quad (10)$$

F3 footing has qz-spring at six points; two of them are located at increased end effect area. The stiffness matrix is given as(s is spring spacing, EI is footing property)

$$K = \frac{EI}{s^2} \begin{bmatrix} \left(1 + \frac{k_{end}s^4}{EI}\right) & -2 & 1 & & & \\ -2 & \left(5 + \frac{k_{mid}s^4}{EI}\right) & -4 & 1 & & \\ 1 & -4 & \left(6 + \frac{k_{mid}s^4}{EI}\right) & -4 & 1 & \\ & 1 & -4 & \left(6 + \frac{k_{mid}s^4}{EI}\right) & -4 & 1 \\ & & 1 & -4 & \left(5 + \frac{k_{mid}s^4}{EI}\right) & -2 \\ & & & 1 & -2 & \left(1 + \frac{k_{end}s^4}{EI}\right) \end{bmatrix} \quad (11)$$

The equation of motion is $M\ddot{z} + Kz = u(t)$ and let $z_1 = z(t)$, $z_2 = \dot{z}$, $z = [z_1, z_2]^T$ then it is described by the nonlinear differential equation of the form.⁽¹⁴⁾

$$\begin{aligned} \dot{z}(t) &= h(z(t), u(t), t) \\ y(t) &= f(z(t), u(t), t) \end{aligned} \quad (12a, b)$$

Now suppose the input is perturbed slightly to become

$u_0(t) + \bar{u}(t)$ and the initial state is also perturbed only slightly. So the corresponding solution may differ from $z_0(t)$ only slightly. In this case, the solution can be expressed as $z_0(t) + \bar{z}(t)$ with $\bar{z}(t)$ small for all t . Under this assumption, we can expand Equation 4.3-6 as

$$\dot{z}_0(t) + \dot{\bar{z}}(t) = h(z_0(t), u_0(t), t) + \frac{\partial h}{\partial z} \bar{z} + \frac{\partial h}{\partial u} \bar{u} + \dots \quad (13)$$

For N-DOF's system,

$$A(t) := \frac{\partial h}{\partial z} := \begin{bmatrix} \frac{\partial h_1}{\partial z_1} & \dots & \frac{\partial h_1}{\partial z_N} \\ \vdots & & \vdots \\ \frac{\partial h_N}{\partial z_1} & \dots & \frac{\partial h_N}{\partial z_N} \end{bmatrix} \quad (14a)$$

$$B(t) := \frac{\partial h}{\partial u} := \begin{bmatrix} \frac{\partial h_1}{\partial u_1} & \dots & \frac{\partial h_1}{\partial u_N} \\ \vdots & & \vdots \\ \frac{\partial h_N}{\partial u_1} & \dots & \frac{\partial h_N}{\partial u_N} \end{bmatrix} \quad (14b)$$

Neglecting higher power of $\bar{z}$ and $\bar{u}$ Equation 13 can be reduced to

$$\dot{\bar{z}}(t) = A(t)\bar{z}(t) + B(t)\bar{u}(t) \quad (15)$$

3.5 Application of LMI Model Reduction

The footing with Qzsimple 1 springs is divided into three parts; two end regions and one mid region. To reduce the order of this model efficiently maintaining its topology, Linear Matrix Inequality(LMI) reduction can be a useful tool. A matrix inequality

$$F(x) = F_0 + x_1 F_1 + \dots + x_n F_n > 0 \quad (16)$$

is called linear matrix inequality, where

$F(x)$: affine function of the real vector $X = [x_1, x_2, \dots, x_n]^T$

$F_0, F_1, \dots, F_n$: real symmetric matrices

X is generalized controllability gramian and Y is generalized observability gramian if they satisfy the

Lyapunov inequalities

$$AX + XA^* + BB^* \leq 0, A^*Y + YA + C^*C \leq 0 \quad (17)$$

Through the coprime factorization process, the structured gramians are obtained

$$\begin{bmatrix} \bar{X}(A - BR^{-1}\bar{D}^*\bar{C}) + (A - BR^{-1}\bar{D}^*\bar{C})\bar{X} + \bar{C}^*\bar{S}^{-1}\bar{C} & \bar{X}B \\ B^*\bar{X} & R \end{bmatrix} \geq 0$$

$$\begin{bmatrix} A - BR^{-1}(B^*\bar{X} + \bar{D}^*\bar{C}) \\ \bar{Y} + \bar{Y}^* \end{bmatrix} \begin{bmatrix} A - BR^{-1}(B^*\bar{X} + \bar{D}^*\bar{C}) \\ A - BR^{-1}(B^*\bar{X} + \bar{D}^*\bar{C}) \end{bmatrix}^* < 0 \quad (18a, b)$$

where $\bar{C} = \gamma C$, $\bar{D} = \gamma D$, $R = I + \bar{D}^*\bar{D}$, $S = I + \bar{D}\bar{D}^*$, and as a result, Equation (19) is obtained.

$$\bar{X} = \bar{X}^* = \begin{bmatrix} \bar{x}_1 & 0 & 0 \\ 0 & \bar{x}_2 & 0 \\ 0 & 0 & \bar{x}_3 \end{bmatrix} \quad (19a)$$

$$\bar{Y} = \bar{Y}^* = \begin{bmatrix} \bar{y}_1 & 0 & 0 \\ 0 & \bar{y}_2 & 0 \\ 0 & 0 & \bar{y}_3 \end{bmatrix} \quad (19b)$$

The controllability gramian and observability gramian shown in Equation (19) are in form of block diagonal matrices. They are intentionally separated into three blocks to keep the topology of each block. Now, each of gramians can be balanced without mixing the states with other blocks. Finally we have three balanced realization corresponding to their region as shown in Figure 10.

〈Fig. 10〉 LMI Model Reduction for Foundation

4. Results and Conclusion

A 3-D finite element model of Cal(IT)2 building was developed with ETABS® and analysis performed to obtain an insight to the modal characteristics(Figure 11). Some material properties are summarized in Table 6.

The shear walls are modeled with “membrane” element to represent in-plane member stiffness. The cast-in-place walls and the slabs are modeled with “shell” element to represent out-of-plane bending stiffness as well as in-plane member stiffness.

Dimensions and material properties of members follow the as-built drawings. Simplified Gazetas boundary condition and fixed boundary condition are considered for comparison. The model has been updated according to measured vibration data. As expected, the model with fixed supporting condition shows higher natural frequencies but the gap is small(Table 7).

The Yucaipa earthquake records are compared to the simulated acceleration and displacement using the FE model at sensor location 10-D of each level(Figure 12, Figure 13). The recorded and simulated acceleration response

〈Fig. 11〉 ETABS® model view from North–West side

〈Table 6〉 Model Material Properties

Element Type	Material	Total Weight (kips)	Num of Pieces
Column	CONC	1764.107	214
Beam	STEEL	65.838	160
Beam	CONC	7528.661	316

〈Table 7〉 Natural Frequencies

mode	Simplified Gazetas Springs	Fixed Support
1st	2.996	3.077(2.7% difference)
2nd	3.411	3.511(2.9% increased)
3rd	4.687	4.718(0.7% increased)

〈Fig. 12〉 General Plan and Sensor Location at point 10–D

(a) Longitudinal Direction at 2nd Level

(b) Longitudinal Direction at Mechanical Level

(c) Transverse Direction at 2nd Level

(d) Transverse Direction at Mechanical Level

〈Fig. 13〉 Recorded vs. Simulated Response

and amplitude match fairly well. In the future study, it will be interesting to observe how the updated supporting condition affects the simulation.

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