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RESEARCH ARTICLE

ANALYSIS OF THE COLLISION EFFECT FOR ADJACENT STRUCTURES IN THE TIME-FREQUENCY DOMAIN

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ABSTRACT

Many reinforced concrete structures are built side by side in our cities. There are very few applications of earthquake joints in adjacent apartment buildings, most of them were constructed completely adjacent. Adjoining buildings or any property should be protected from damage or collapse during construction, restoration and demolition work. In case of the static condition, protection must be supplied for foundations, walls and roofs. We have already observed many building collapses in Turkey with a great number of life lost. For the constructed buildings, there is a need to understand the structural system of an existing building to manage risk. In case of earthquakes like dynamic loading, earthquake joints are essential. The Turkish Building Earthquake Code (TBEC) 2018 requires calculations of vertical joints for some types of structures. Earthquake joints should be left in the gap calculated for structures of some height. It is thought that this gap will prevent the structures from colliding. However, it has been the subject of research how the performance of the building will change when it is produced adjacent or when floor offsets cause collisions at earthquake joints. In this study, structures of different heights were analyzed in the time domain for specific earthquake motions. Collision situations of more than two buildings are discussed by making a street model. Structural performances of discrete and adjacent structures obtained from nonlinear analysis were compared. The motion parameters obtained from the structures were analyzed in the time-frequency domain by wavelet and Hilbert transforms. To determine the effects of earthquake joints, condition assessment and damage detection studies were carried out and the sufficiency of seismic joints was discussed.

Keywords: Adjacent structures, Collision impact, Vertical seismic joints, Wavelet transform, Hilbert Huang Transform

1. INTRODUCTION

In the section related to earthquake joints in the Turkish earthquake code, it is stated that vertical earthquake joints should be placed at certain distances between adjacent structures built depending on the height. The minimum joint space to be left in by article 2.10.3.2 will be at least 30 mm up to 6 m height and at least 10 mm will be added to this value for every 3 m height after 6 m. While these joints were left in the structures built after the earthquake, many existing structures were built completely adjacent to each other before the earthquake. It has been a subject of research how the structural behaviors change if the floor displacements cross the earthquake joints and cause collisions as a result of the oscillations in opposite directions at the same time in such adjacent buildings or buildings with different periods. The adequacy of earthquake joints and the behavior of fully adjacent structures were investigated by nonlinear analysis in the time domain under the effect of the eleven earthquake recordings through the analytical model. Changes in structural performances are compared when the building is separate and adjacent. Results were analyzed in time-frequency domain by wavelet analysis and the Hilbert Huang transform method.

The main study on collision in adjacent buildings was carried out by Anagnostopoulos [1]. In the analysis, structures are modeled with single degree of freedom systems and collisions are simulated with the help of a linear viscoelastic model of impact force. Multi-degree of freedom (MDOF) models, with the mass of each floor collected at the floor level, are used to analyze in more detail the earthquake-

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related collision in buildings of different heights. Among analytical analyzes, collisions are modeled as nonlinear impact between a single degree of freedom oscillator and a rigid barrier based on the nonlinear Hertz collision model used by Davis [2]. Taking the collision effect into account in the nonlinear field gives more realistic results in calculations. Maison and Kasai [3] established the equation of degrees of freedom of movement for different structure collisions. With this theory, he studied the behavioral characteristics of a 15-story building in the computer environment. Filiatrault et al. [4] conducted vibration table tests for collisions between adjacent three and eight-story steel frames in the time history of the 1940 El-Centro earthquake and the experimental results were compared with the predictions given by the two computer programs. Papadrakakis and Mouzakis [5,6], by subjecting to sinusoidal and random movements, conducted shake table experiments for collision between two-storey reinforced concrete buildings with zero spacing and their experimental results were compared with the estimates made by the Lagrange multiplier method.

Other approaches have been proposed to determine the relative displacement between two structures during earthquakes. For example, Stavroulakis and Abdalla [7] used the pseudo-static method, Filiatrault and Cervantes [8] used nonlinear time history analysis, and Lin [9] proposed the stochastic method of random vibration. Despite all these differences in ideas, there is consensus that current building code provisions for seismic collision are very protective. The spatial ground motion effect on impacts was evaluated by Jeng and Kasai [10] and Hao and Zhang [11]. In most of these studies, a shock absorber spring model with linear stiffness or bilinear stiffness was used to model the impact force between buildings. However, the actual impact forces between structures or between various components of a structure are generally not linear. Pantelides and Ma [12] examined the collisions between a single degree of freedom and damped structure and a rigid obstacle with ground motions of the 1940 El Centro, 1971 San Fernando, 1989 Loma Prieta and 1994 Northridge earthquakes. Chau and Wei [13] examined nonlinear collisions between two single degrees of freedom oscillators. Rahman et al. [14] examined the collision of two 12-storey and 6-storey buildings with different dynamic properties, considering the ground properties. Muthukumar [15] revealed that in the 1999 Kocaeli earthquake, a six-storey building in Gölcük collided with the neighboring two-storey building, and the column on the third floor of the six-storey building was severely damaged and the two-storey building collapsed due to shear forces. Gong and Hao [16] analyzed the torsional effects between symmetrical and non-symmetrical systems that are subjected to bidirectional ground motion. Wang and Chau [17] modeled the torsional collision between two unsymmetrical buildings using the nonlinear Hertz model technique. As a result of the studies, he stated that the torsion effect is generally complex compared to the translational effect. Jankowski [18] studied a detailed three-dimensional pounding response analysis of Olive View Hospital main building using the finite element method (FEM) with nonlinear model material behavior with stiffness degradation of concrete during the San Fernando earthquake in 1971. Pant et al. [19] presented a three-dimensional simulation of seismic collisions between reinforced concrete moment-resistant frame buildings, considering the material and geometric nonlinear structures. Çetinkaya [20] analyzed the collision of two neighboring buildings with different stiffness for 4 different spring models and compared the results. He concluded that the model in which the interaction between buildings is seen most clearly is the Hertz (nonlinear elastic spring) model. Mahmoud et al. [21] examined the impact of soil elasticity on these two structures as well as the collision of two non-linear structures of equal height under the effect of an earthquake. As a result of the analysis, it has been observed that the cyclic and horizontal movements of the ground affect the collision of two buildings. Mate et al. [22] presented a comparative study of various existing linear and nonlinear simulation models for collision in three adjacent single degrees of freedom and multi-degree of freedom linear elastic structures. This study showed that the impact result depends on the ground motion characteristics and the relationship between the first periods in buildings. Akköse and Sunca [23] evaluated the seismic performance of a train station building in the 2011 Van earthquake by nonlinear time history analysis and the results have been compared with and without pounding effects. Beyen [24] stated that the results of building diagnosis can contribute to the situation analysis of existing structures in structural health checks after an important earthquake. Updating analytical building models and identifying them by the facts are important for determining the post-earthquake situation in adjacent structures. Kamal and Inel [25] investigated the effects of pounding on seismic behavior of 5, 8, 10, 13 and 15 storey RC buildings and showed that significant changes may occur in the building displacement demands due to the collision of the mid-rise RC neighboring buildings with the insufficient seismic gap. As a result of the study, the displacement factor for medium-rise RC buildings was proposed. Cayci and Akpinar [26] evaluated pounding effects on typical building structures considering soil-structure interaction. They studied nonlinear time history analysis of 4, 8, 12 and 16 story buildings with 15 different ground motions and they showed that the minimum distance required to avoid collision seems not to be sufficient.

2. COLLISION MODEL

In this model, to represent the relationship between the force of impact and displacement, the Hertz law of contact has been utilized. Nonlinear elastic spring is activated by closing the space (d) between buildings. The collision force is represented by u (t) = u_i (t) - u_j (t) as follows;

$$F_C = k_G[u(t) - d]^{3/2} \quad u(t) - d > 0 \quad (collision\ situation) \qquad (2a)$$

$$F_C = 0 \quad u(t) - d \leq 0 \quad (noncollision\ situation) \qquad (2b)$$

Here; ui (t) and uj (t) are the relative displacements of neighboring buildings in the same direction, d represents the space between two buildings, k_G is the nonlinear elastic spring constant, F_c is the impact force. The mass density ρ and radius R_i approximation of colliding structures can be found from the equation below. [Goldsmith (1960)]:

$$R_i = \sqrt{\frac{3m_i}{4\pi\rho}}\ , \qquad i = 1{,}2 \qquad (2c)$$

Nonlinear spring tension k_h depends on the material properties and radii of colliding structures according to the formula: where h_1 and h_2 are the material parameters defined by the formula:

$$k_h = \frac{4}{3\pi(h_1 + h_2)}\left[\frac{R_1 R_2}{R_1 + R_2}\right]^{1/2} \qquad (2d)$$

$$h_i = \frac{1 - \gamma_i}{\pi E_i} \quad i = 1{,}2 \qquad (2e)$$

Here, γ_i and E_i are the Poisson's ratio and the modulus of elasticity, respectively. The coefficient K_h depends on the material properties and geometry of the colliding objects.
As the structural strength in the calculation, the bulk density is taken as ρ = 2500 kg/m^3, Poisson's ratio γ_i = 0.2, elasticity modulus $E_i = 2.8\text{x}10^{10}$ N/m^2 and $k_G = 1.13\text{x}10^9$ N/m$^{3/2}$ for concrete to concrete impacts based on numerical simulation suggested by Jankowski [27].

3. ANALYTICAL STUDIES

Four, five and six-storey structures were modeled in accordance with TS500 [28] and TBEC (2018) [29], the columns were selected as 30x60 cm^2, and the beams as 25x50 cm^2 with C25/30 class concrete. The effective section stiffness is 0.7 for columns and 0.35 for beams. Spans in *x* and *y* directions in plan are selected as 3m and the floor height is 3m as shown in Figure 1. Analytical structures according to the earthquake specifications, the earthquake joints were located completely adjacent and singular. With the 11 earthquake recordings, numerical model was analyzed in a nonlinear fashion in the time domain and damage parameters were determined. Damage detection was performed using the wavelet transform and Hilbert Huang transform, based on deformation distributions in the joints and acceleration records obtained through the structural observation points. In addition, the columns connected to the foundation in the structure were modeled as built-in and fixed support and changes in the distribution of damage were also observed for the same earthquake loading.

First fundamental period of the structure, soft story, weak story, torsional irregularity check, limitations of relative story drifts and second order effects were performed in accordance with TBEC (2018) [29] and the results were acceptable laying in the appropriate ranges. The modal parameters of the structure were determined by using modal analysis approach within the finite element package.

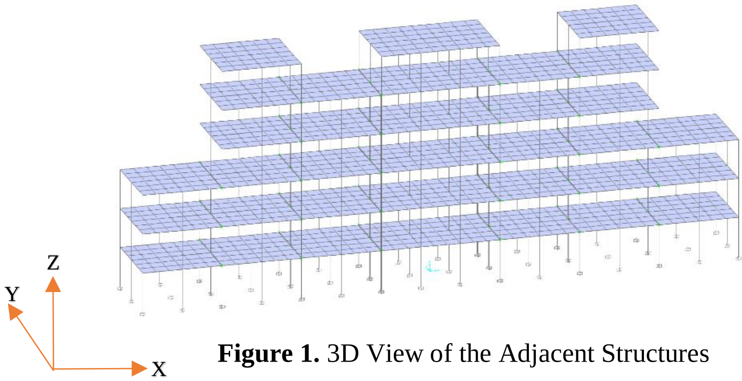

Figure 1. 3D View of the Adjacent Structures

3.1. Determination of Earthquake Performance of The Adjacent Buildings by Applying Nonlinear Method for A Series of Earthquake Recordings.

Results of the nonlinear case tabulated and plotted in Figures 1 and 2 were obtained for the identified modal parameters of the structural system by the SAP2000 package for the strong earthquake loading. As an acceleration record, 11 different earthquakes from different recording stations were scaled and used including the data of the Yarımca station recorded during Kocaeli earthquake 1999. The earthquake archive of this study is listed, and their spectral graphs are shown in Figure 2.

Result ID	Spectral Ordinate	Record Seq. #	MSE	Scale Factor	Tp(s)	D5-75(s)	D5-95(s)	Arias Intensity (m/s)	Event	Year	Station	Mag	Mechanism
1	SRSS	6	0.0926	1.4294	-	17.7	24.2	1.6	Imperial Valley-02	1940	El Centro Array #9	6.95	strike slip
2	SRSS	15	0.0583	2.3195	-	10.7	30.3	0.6	Kern County	1952	Taft Lincoln School	7.36	Reverse
3	SRSS	20	0.2862	1.8676	-	6.8	19.4	0.5	Northern Calif-03	1954	Ferndale City Hall	6.5	strike slip
4	SRSS	30	0.2584	1.9328	-	2.4	7.5	0.9	Parkfield	1966	Cholame - Shandon Array #5	6.19	strike slip
5	SRSS	36	0.3868	2.8865	-	25.0	49.3	0.2	Borrego Mtn	1968	El Centro Array #9	6.63	strike slip
6	SRSS	57	0.8564	2.9471	-	10.6	16.8	1.0	San Fernando	1971	Castaic - Old Ridge Route	6.61	Reverse
7	SRSS	95	0.3937	1.5033	-	4.9	10.6	2.0	Managua, Nicaragua-01	1972	Managua, ESSO	6.24	strike slip
8	SRSS	125	0.4961	2.0327	-	2.5	4.9	1.2	Friuli, Italy-01	1976	Tolmezzo	6.5	Reverse
9	SRSS	126	0.0789	0.6988	-	5.6	7.0	5.7	Gazli, USSR	1976	Karakyr	6.8	Reverse
10	SRSS	138	0.1395	2.8759	-	14.6	19.5	0.3	Tabas, Iran	1978	Boshrooyeh	7.35	Reverse
11	SRSS	1176	0.4872	0.8834	4.949	7.0	15.1	1.3	Kocaeli, Turkey	1999	Yarimca	7.51	strike slip

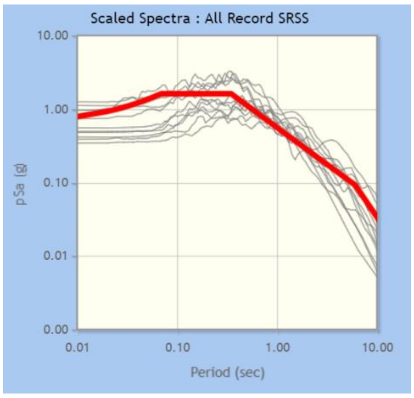

Figure 2. Earthquake Acceleration Records and Spectral Graphs (Peer Ground Motion Database)

Table 1. Undamaged (Modal) and damaged (Modal-NL) Jointed Structure Parameters

OUTPUTCASE	STEP TYPE	STEP NUM	PERIOD	FREQUENCY	CIRC FREQ	EIGENVALUE
			Sec	Cyc/sec	rad/sec	rad2/sec2
MODAL	Mod	1	0.707	1.414	8.887	78.973
MODAL	Mod	2	0.690	1.450	9.111	83.003
MODAL	Mod	3	0.690	1.450	9.111	83.003
MODAL	Mod	4	0.571	1.751	11.002	121.049
MODAL-NL	Mod	1	0.711	1.406	8.837	78.088
MODAL-NL	Mod	2	0.693	1.442	9.062	82.117
MODAL-NL	Mod	3	0.693	1.442	9.063	82.144
MODAL-NL	Mod	4	0.574	1.743	10.954	120.001

In Table 1 the period of the first mode for the *x* direction is slightly increased for the undamaged model from 0.707 sec to 0.711 sec in case of the damage.

When the plastic hinges of the columns in the structure were examined, in Figure 3 it was observed that several elements surpassed the collapse prevention performance level to one step ahead collapse at the first floor. The reference parameters of the study model are assigned based on TBEC (2018) values and lumped plasticity approach has been adopted.

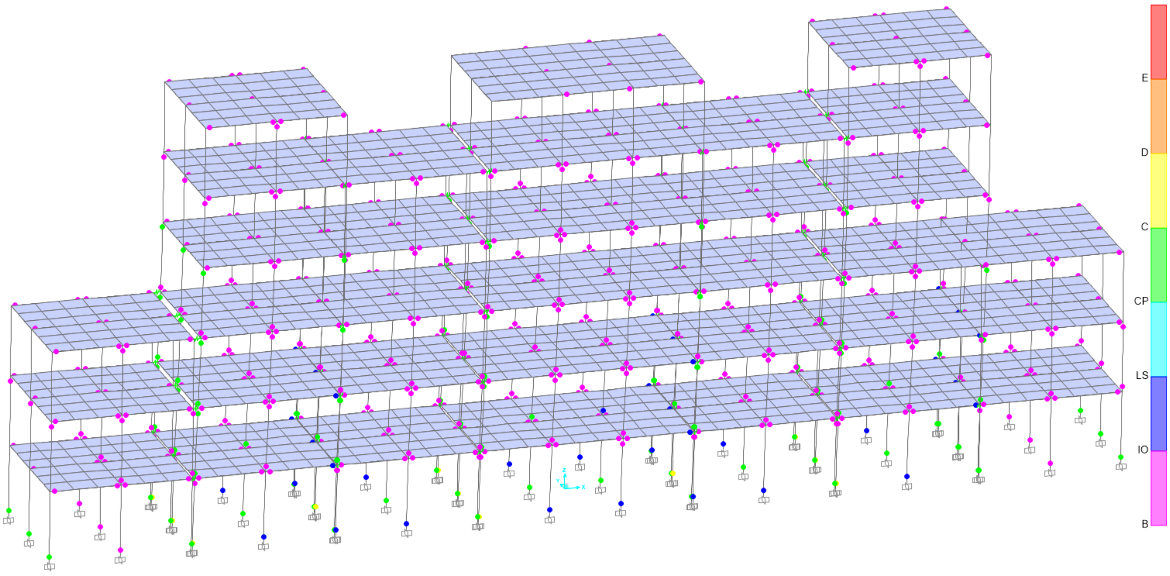

Figure 3. Plastic Hinges Graph for Joint Structures. Result of the Joint Conditions in case of the Strongest Earthquake (IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention, C: Collapse)

The period of the dominant mode for the *x* direction increased from 0.707 sec in the undamaged model to 0.711 sec in the damaged model. The frequency change in the discrete structure is lower than in the joint structure. Looking at the order of damage, it is seen that from Figure 4 the level of performance in structural elements does not change significantly. From such a result, it can be concluded that the adjacent spans of the earthquake joints proposed in the earthquake regulation is sufficient for the structural system.

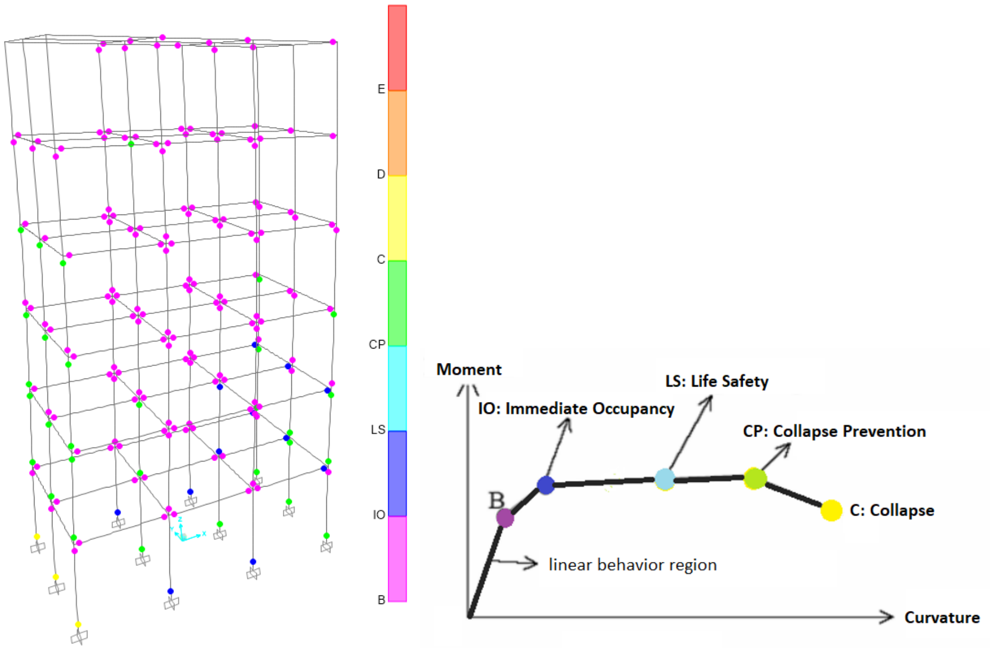

Figure 4. Plastic Hinges Graph for Single Structure Result of Analysis in case of the Strongest Earthquake.

Table 2. Undamaged (Modal) and Damaged (Modal-NL) Single Structure Parameters

OUTPUT CASE	STEP TYPE	STEP NUM	PERIOD	FREQUENCY	CIRC FREQ	EIGENVALUE
			Sec	Cyc/sec	rad/sec	rad2/sec2
MODAL	Mod	1	0.707	1.414	8.887	78.973
MODAL	Mod	2	0.521	1.920	12.066	145.591
MODAL	Mod	3	0.512	1.951	12.260	150.307
MODAL	Mod	4	0.231	4.334	27.233	741.660
MODAL-NL	Mod	1	0.711	1.406	8.837	78.088
MODAL-NL	Mod	2	0.522	1.915	12.031	144.750
MODAL-NL	Mod	3	0.514	1.944	12.214	149.170
MODAL-NL	Mod	4	0.232	4.318	27.131	736.094

When we consider the structure that is insufficient according to the regulation by taking the earthquake joint 1 cm, the structure is damaged at an advanced level in the face of the same earthquake forces in Figure 5 and Figure 6. It is understood that the distances specified in the regulation are sufficient and necessary when compared with the structure where sufficient adjacent spans are not applied.

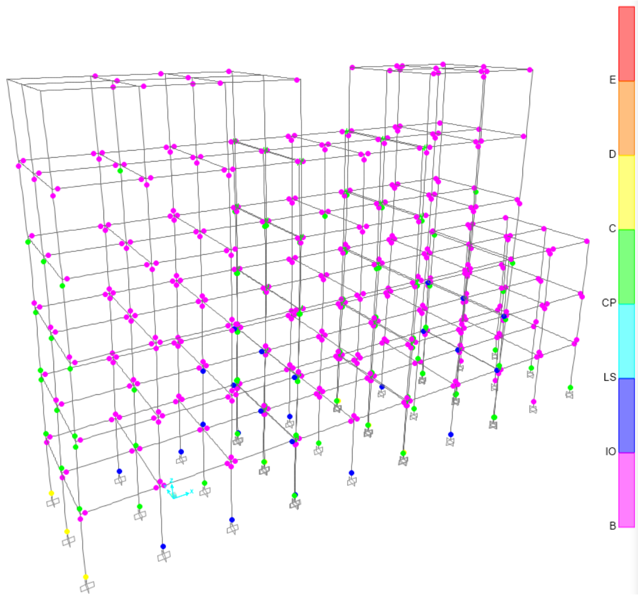

Figure 5. Plastic Hinge Graph for Structures Constructed with Sufficient Adjacent Spans. Result of Analysis in case of the Strongest Earthquake.

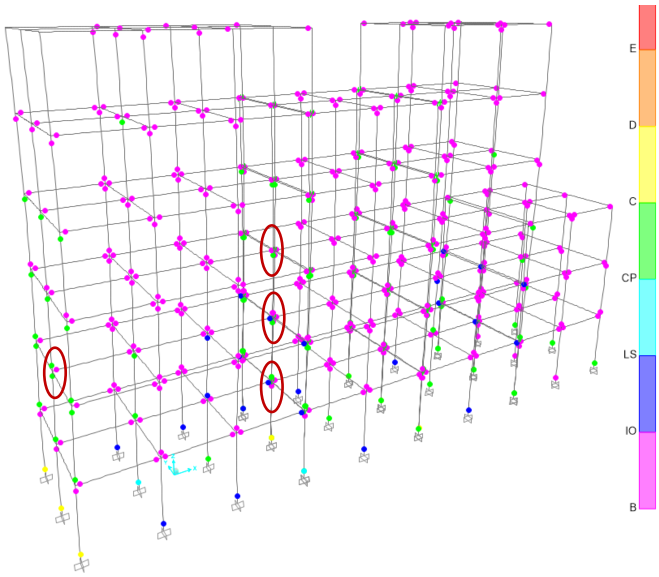

Figure 6. Plastic Hinge Graph for Structures that don't have Enough Adjacent Spans. Result of Analysis for the Strongest Earthquake.

Especially when the first structure is considered, damage levels that reaches to the collapse prevention zone are observed marked in Figure 6. From the structures left with insufficient joints, the columns in the structure adjacent to the first building are increased from 30x60 cm^2 to 100x100 cm^2, resulting in a similar distribution of damage when looking at the interaction of the rigid structure.

The rigidity of the building adjacent to the structure, where the joint gap is insufficient as seen in Figure 7, somewhat reduces the damage levels, but as a result, structural performance levels cannot reach the predicted levels. Column shear force increases from 152.45 kN to 161.71 kN in the structure where there is not enough space. The base shear force increases from 361.05 kN to 372.85 kN. As a result, the column shear forces observed in the collision increased by approximately 6%, and the base shear force increased by 3%.

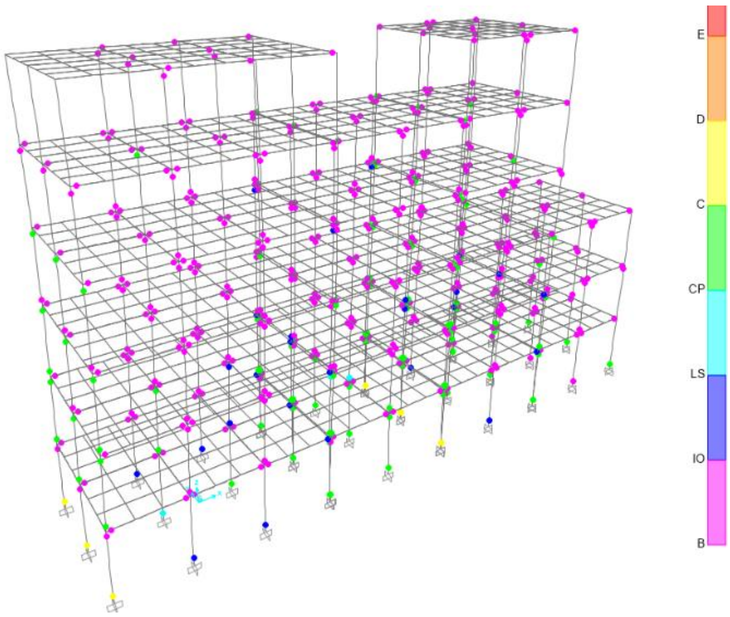

Figure 7. Hinge Graph for a Rigid Neighboring Structures That don't have Enough Adjacent Spans. Result of Analysis for the Strongest Earthquake Case

3.2. Damage Diagnostic Work by Wavelet Analysis Method Over Damaged and Undamaged Parameters

The damage is mainly concentrated in columns as seen in Figure 7. For discrete time-frequency changes in operational modal parameters, discrete Daubechies (Db4), Morlet, Mexican Hat, and Symlet wavelet filters were tested to detect the damage by examining the differences between the two conditions. Details of the theory and implementation of Time-Frequency analysis can be found in following studies Sak and Beyen, Beyen [30, 31, 32]. The necessary codes for this are written in MATLAB. As a result of the analysis, Db4 wavelet models were found to be suitable and sufficient for the analysis of the structures. Here, based on the damaged and undamaged records of one of the damaged elements, we look at the noticeable differences in the wavelet and Hilbert Huang transformation.

As can be seen from the graphs in Figure 8, the frequency order changes in the wavelet spectra at the time of damage formation. And in the Figure 9 Hilbert Huang transformation, the frequency change of the undamaged element is significant, while in the damaged element it changes depending on time.

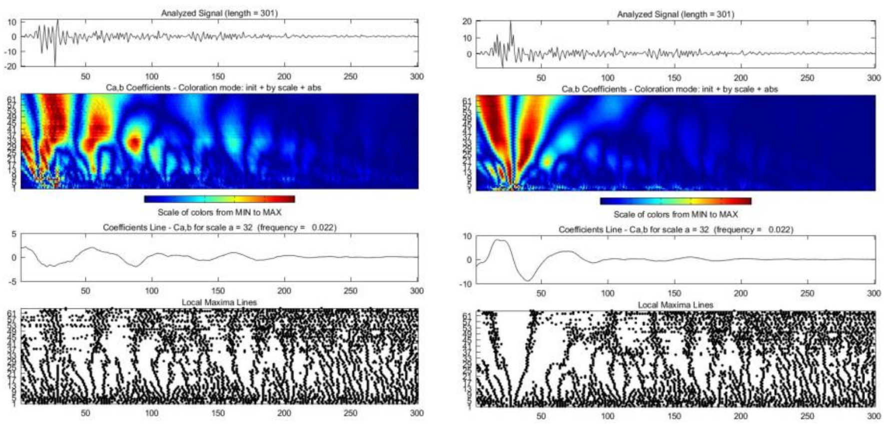

Figure 8. Wavelet Transformation of an Element Undamaged (left) and Damaged (Right) for an intensive Earthquake

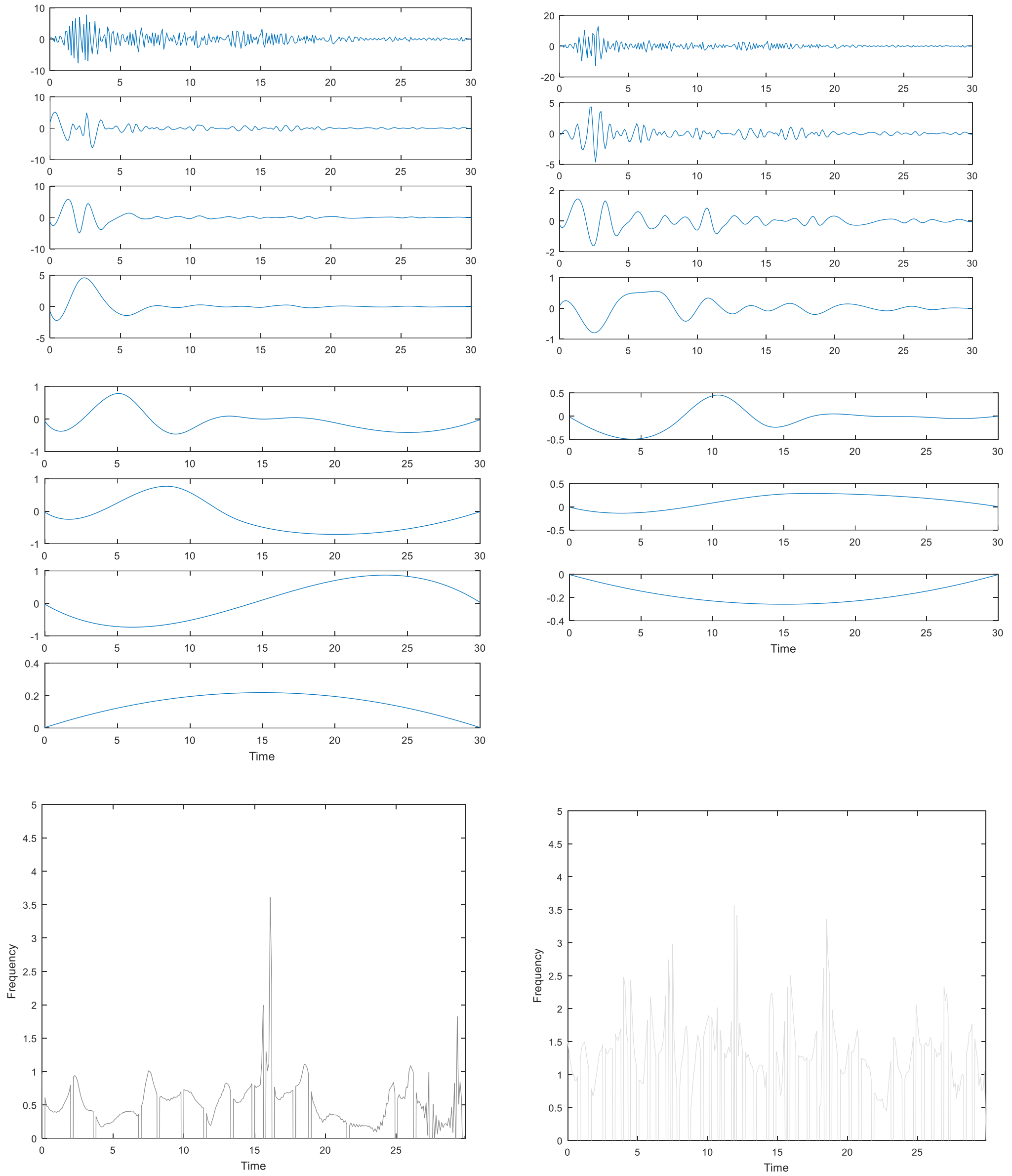

Figure 9. Hilbert Huang Transformation of Undamaged (Left) and Damaged (Right) Elements

4. RESULT

As revealed in this study, the vertical earthquake joints determined in TBEC (2018) [29] are necessary and sufficient for adjacent structures. In structures where earthquake joints are not applied, structural performance levels cannot be encountered. In adjacent buildings, each building's rigidity in line reduces structural damage to some extent, but the target performance level cannot be reached. For this reason, leaving vertical earthquake joints in the adjacent span determined in the regulation significantly affects the earthquake performance of the buildings.

Damage detection with a wavelet and Hilbert Huang transforms can be used as effective methods in structural health monitoring autonomous systems. Damaged elements and undamaged elements can be detected based on the frequency content that changes over time in the spectra.

In the earthquake regulation that will be updated in future, structural health monitoring strategies can be used, which give high resolution in the time-frequency domain as declared in this study.

CONFLICT OF INTEREST

The authors stated that there are no conflicts of interest regarding the publication of this article.

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