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# The Effect of Vertical Component of Earthquake on Continuous and Monolithic Frame Bridges

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**Abstract**

Performance of structures during recent earthquakes shows that the effect of vertical component of earthquake (VCE) could be considered as one of the main causes of bridges collapse. In most of bridge design codes, for seismic analysis of bridges, VCE is not taken into account or a distinguished method isn't presented for assessment of VCE. In the present work, the effect of VCE on two existing bridges, one with continuous deck and the other with monolithic frame system, was studied. The first model consisted of a pre-stress bridge in which the superstructure was connected rigidly to piers. The super structure consisted of 3 spans with length of 16, 48 and 16 *m*. The end of side slabs was put on abutments. The second model was a bridge with steel deck and concrete piers. The bridge superstructure was composed of I girder beams and in-place concrete slab. The beams were placed on 3 piers located 24 *m* far from each other, continually. In both models, the effect of VCE was studied considering the 3 acceleration of Tabas, Northridge and Kobe earthquakes and using linear and nonlinear time history and spectrum analysis on 3D models. In each analysis, the model was analyzed considering the 3 component and 2 horizontal components of earthquakes separately. The ratio of the difference of results in two analyses to the result of bridge response under its weight (DL) was compared. Through this method the amount of VCE effect on affected elements (according to statistical system of bridge) was found out.

***Key word:*** Monolithic bridge, Continuous frame, Earthquake, Vertical component.

## 1. Introduction

Most of bridge designers suppose that the effect of vertical component of earthquake (VCE) doesn't have any important effect on bridges. This may be due to the point that the codes do not point to these effects directly. However, some codes try to consider the effect of VCE by increasing or decreasing the amount of dead load. This method has been used in seismic guidance of AASHTO [1]. Load factors of 0.8 and 1.2 of dead load (increasing and decreasing of 20% of dead load) are considered by this code. However, these factors do not consider the effects of earthquake magnitude, fault distance and bridge types.

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The purpose of this study is to investigate the effect of VCE in two kinds of bridges. Therefore, two different bridges, one with monolithic frame system and the other with continuous superstructure were modeled and in each model, linear and nonlinear dynamic analysis, first without entering the VCE and second by considering the VCE, was done and results of the analyses were compared.
Sadeghvaziri and Fouch [2] carried out the first analytical studies on the effect of VCE on structures. They developed a uni-axial flexural model in which the columns were represented with an assemblage of plane stress and bar elements to model the concrete and reinforced steel. The results revealed that uncoupled variation of lateral load, leads to a hysteresis response different from the coupled variation of lateral load. Furthermore, the lateral force displacement curve showed negative energy regions, which was a consequence of axial deformations.

Broekhuizen [3-4] investigated the effect of vertical acceleration on pre-stressed concrete bridge decks. By assuming the 1-g upward acceleration, it was found that allowable tensile stresses in the deck could be exceeded. However, the author concluded that since the acceleration was instantaneous, the cracking mechanism would not have time to start and any tensile cracks that did form would be controlled by the continuous reinforcement in the deck.

Elnashai [5] in assessment of the fractures of bridges in Kobe and Northridge earthquakes concluded that the reason of column fracture in most of bridges was because of increasing in axial load concluded by VCE. The results of this study showed that the compression failure in columns was the most important factor behind collapses of bridges.

Yan Xiao and AsadEsmaeily-G [6] tested 6 large scale circular reinforced concrete columns under different loading conditions. They showed that the axial force level and path play significant roles in the behavior of the columns. Their experimental results concluded that an increase in axial compression loads leads to increase in the flexural capacity, but to decrease in the ductility.

Button and Cronin [7] investigated the effect of VCE on 6 bridges with different static systems. Their study consisted of linear spectrum analysis and linear and nonlinear time history analysis. To consider the effect of VCE on bridges, they recommended some load factor according to bridge situation and fault distance.

## 2. Vertical Ground Motion Characteristics

In near field regions (D < 10-15 km) and in large earthquakes, ground motion loses its stable and predictable behavior. In these regions, time domains between acceleration changing decreases and speed and deformation magnitude increases. Also in short period and in both stone and soil regions, vertical spectra can exceed horizontal spectra (Fig 1). However, in far fault regions, vertical spectra decreases and in short periods it would be lower than horizontal spectra (Fig 2).

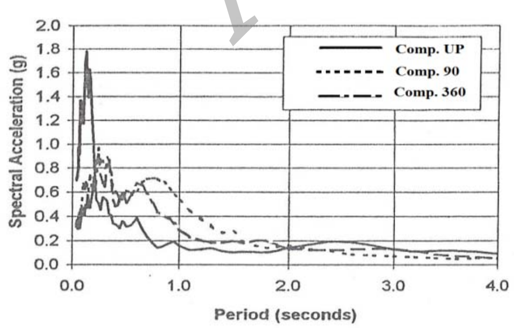



Fig1: Response spectra for 1994 Northridge-Arleta record

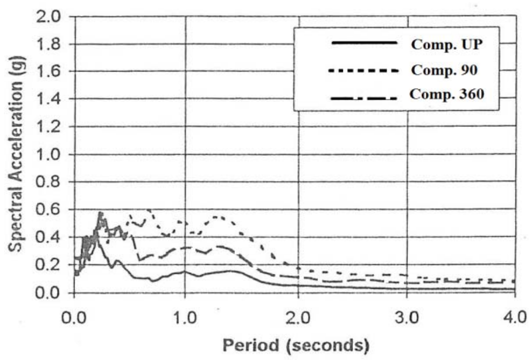



Fig2: Response spectra for 1992 Landers-Yermo record

Silva [8] stated the general behavior of vertical spectra acceleration in relation to horizontal spectra acceleration. According to Silva's research, the ratio of vertical spectra to horizontal spectra is as follows:

In zero periods, the ratio of V/H is about 0.67. At period about 0.1 second, this ratio increases rapidly and in periods higher than 0.2-0.3 second it decreases. In earthquakes with magnitude of 6.5, the ratio of V/H has the maximum amount of 1.1 (stone regions) and 1.9 (soil regions). In earthquakes with magnitude of 7.5, these ratios increase to 1.3 (stone regions) and 2.6 (soil regions) [8].

## 3. Studied Models

To investigate the effect of VCE on frame bridges with continuous deck and monolithic frame system, two models of existing bridges were made.

The first model consisted of a 3-span bridge with length of 80m and width of 11m. All elements of bridge were of concrete materials. Superstructure was connected to piers rigidly by pre-stressed tendons. Superstructure consisted of two pre-cast box sections with length of 2.4 m and width of 5.5 m. Top slab thickness was 0.22 m in all of the bridge length but the lower slab had a variable thickness of 0.58 m at mid-span to 0.18 m on piers. The superstructure was pre-stressed by 12V13 cables with $F_{pu}$ = 1880 N/mm$^2$. The concrete compressive strength was 300 kg/cm$^2$ on cylindrical specimens.

Substructure consisted of 4 pre-stressed columns and two abutments at each end of the bridge. Pier connection to foundation was pin connection.

The second model was continuous bridge with composite superstructure and concrete columns. Superstructure consisted of 7 I-girders, which were 0.9 m far from each other. 4 hammer shape concrete columns with distance of 24 m supported the deck. Columns connection to foundation was fixed connection.

## 4. Used Earthquakes

In each analysis, the acceleration records of 3 earthquakes (Tabas, Kobe and Northridge) were used. Table 1 shows the PGA of these records in each direction. The acceleration records were scaled based on code 2800 [9-10]. Scaled acceleration records used for nonlinear and linear time history analysis and their spectra used for response spectra analysis.

Table 1: The PGA of earthquakes

| | X direction | Y direction | Z direction |
|---|---|---|---|
| Tabas | 0.835 g | 0.851g | 0.688g |
| Kobe | 0.509g | 0.503g | 0.371g |
| Northridge | 1.58g | 1.29g | 1.23g |

## 5. Bridges Modeling

In the present study, 2 models were made for each bridge. For nonlinear analysis, a complete 3D model and for response spectra analysis, a simpler model was used.

### *5.1. Modeling for Nonlinear TH Analysis*

The bridges were modeled in ANSYS finite element software. To model the first bridge, solid, shell and link elements were used to model columns, superstructure and pre-stressing tendons respectively. Solid and shell elements were 4-node elements with capability of modeling nonlinear behavior of concrete. However, the elements do not consider the large deformation effects. Since the thickness of deck was variable near the columns, finer mesh was used to make a better model in these regions. To enter pre-stressing effects in model, link elements were used between superstructure and substructure elements. Initial strain relative to pre-stressing force was applied to each element. Fig 3 shows the model of first bridge. The stress-strain diagram of link elements was supposed to be a bilinear diagram. Furthermore, a druger-pruger criterion was used to model the nonlinear behavior of concrete.

In second bridge, solid element and link element were used to model concrete columns and rebars, respectively. Because of concrete cracking near piers, slab stiffness was neglected in deck modeling. So the superstructure was modeled by beam elements. To have a monolithic deck, transverse beams were used perpendicular to major beams. Fig 4 shows the finite element model of bridge.

Bilinear stress-strain diagram and druger-pruger criterion was used for rebar and concrete columns, respectively. Beam elements were supposed to have linear behavior.

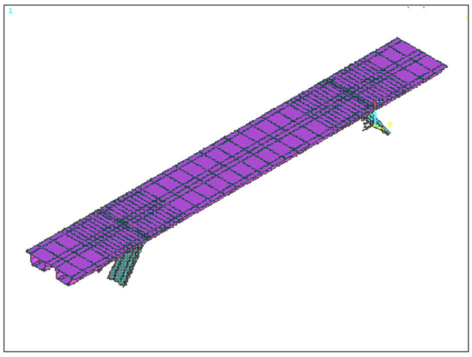


Fig3: Finite element model for monolithic frame bridge

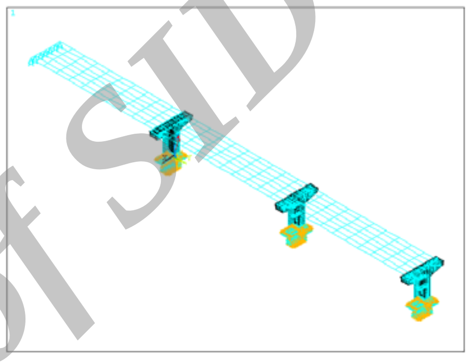
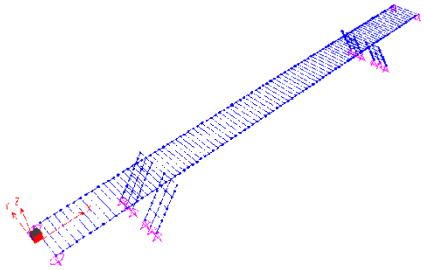


**Fig4:** Finite element model for continuous frame bridge

### *5.2. Modeling for linear Response Spectra Analysis*

To investigate the combination methods in response spectra analysis and control of Nonlinear TH analysis, response spectra (RS) analysis was performed on each of bridges. SRSS, 100%+30% and 100%+40% combination methods were used in RS analyses. The damping ratio of models was supposed to be 5 percent. Piers and deck of each bridge modeled using 3D frame elements. To investigate the correction of modeling of first model, another model was made by shell elements. Since the periods of models in 3D complete models and 3D simpler models were equal, the models were accepted. Fig 5 and Fig 6 illustrate the models of bridges in R.S analyses.



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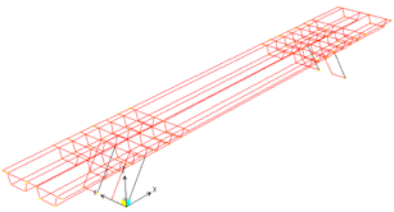

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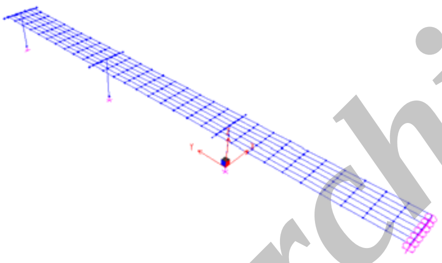
Fig 5: Monolithic bridge models for R. S analysis
a: Frame elements, b: Shell elements



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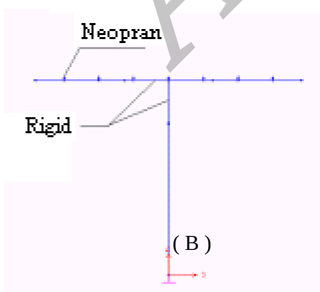


( B )

Fig 6: Continuous bridge models for R. S analysis
a: Bridge model b: Pier detail

## 6. Nonlinear T.H Analyses Results

To investigate the effect of VCE on the above-mentioned bridges, each model was first analyzed under 2 horizontal components and then analyzed under 3 components of earthquake. The results show that in parts of the bridge where dead load has important effect, VCE can influence significantly. In this study, the effect of VCE investigated by comparing the consequent results to results obtained from bridge analysis under its weight. So first bridges analyzed under 3 components and 2 components of earthquakes separately and then the difference of results divided to bridge analysis results under its weight. This ratio is shown with symbol of (3-2)/DL in Tables 2 and 3.
Table 2 shows some of the obtained results of prestressed model. The results show that VCE can increase axial stress about 33% of stresses consequent by dead load at bottom of column. This increasing is about 40% dead load at mid-height and top of column. Furthermore, the results show VCE can cause an increasing of 40% axial stresses and 70% shear stresses in deck. Also the results show that VCE can cause 85% increasing in vertical deflection. However, VCE does not have an important influence on horizontal deflections.
Table 3 shows the results of TH analysis on continuous bridge. The results show that the effect of VCE on continuous bridge is lower than monolithic bridge. The most important effect of VCE on second model is in mid-span moment in which VCE can cause 37% increasing in moments. Also the results show ±20% variation in axial stresses of piers and ±16% variation in superstructure shear.
The results show that VCE can increase the vertical deflection about 20% deflections consequent by dead load. However, similar to the first model, the effect of VCE on horizontal deflection is negligible.



Table 2: Variation of stresses in monolithic model because of VCE

| | (3-2)/DL (%) | | |
|---|---|---|---|
| | Tabas | Kobe | Northridge |
| Axial stress at bottom of column | 2 | 9.2 | 32.3 |
| Axial stress at mid-height of column | 41.7 | 22.6 | 12.75 |
| Axial stress at top of column | 43.5 | 11.6 | 35.9 |
| Axial stress of deck on piers | 9.3 | 6.7 | 44.7 |
| Axial stress of deck at mid-span | 16.4 | 16.2 | 35.2 |
| Shear stress of deck on piers | 32.9 | 12.3 | 69.7 |
| Shear stress of deck on abutment | 46.9 | 24.7 | 40 |
| Vertical deflection of deck at mid-span | 32 | 12 | 85 |

Table3. Variation of Stresses in Continuous Model Because of VCE

| | (3-2)/DL (%) | | |
|---|---|---|---|
| | Tabas | Kobe | Northridge |
| Axial stress at bottom of column | 20 | 13.75 | 22.5 |
| Axial stress at top of column | 11 | 15 | 12.25 |
| Shear of deck on piers | 6 | 5.6 | 16 |
| Positive moment of deck | 7 | 14 | 37 |
| Negative moment of deck | 5.2 | 5.1 | 5.4 |
| Vertical deflection of deck at mid-span | 4 | 5.5 | 19.8 |

## 7. R.S Analyses Results

Table 4 shows the obtained results from R.S analysis on first model. The results show that the combination method 100%+40% is more conservative than SRSS and 100%+30% methods. Results show that VCE can increase the moment of mid-span of deck about 30%. This increase is about 20% deck to pier connection region. These results were about 30% and 38% from TH analysis.

The variation of shear on abutments is about 75% in Kobe and Northridge earthquakes and 20% in Tabas earthquake. These results are about 20% on piers. Furthermore, the results show that axial force increases about 10% at mid-span and 27% at deck to pier connection regions. Also the axial force and moment increase about 20% and 10% in column, respectively.

Table 4. Variation of Loads in Monoilthic Model Because of VCE From R.S Analysis

| | Northridge | | | | Kobe | | | | Tabas | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | TH | SRSS | 100+40 | 100+30 | TH | SRSS | 100+40 | 100+30 | TH | SRSS | 100+40 | **100+30** |
| Moment of deckat mid-span | 28.72 | 27.04 | 27.46 | 27.35 | 3.69 | 30.78 | 31.21 | 31.10 | 13.14 | 12.99 | 13.60 | 13.42 |
| Moment of deck on piers | 38.81 | 18.41 | 18.13 | 13.57 | 22.36 | 20.32 | 20.58 | 15.43 | 14.88 | 2.99 | 9.52 | 7.15 |
| Deck shear on abutment | 109 | 71.76 | 70.74 | 54.20 | 208.0 | 77.40 | 75.36 | 56.51 | 26.58 | 7.86 | 20.40 | 15.32 |
| Deck axial load at mid-span | 10.65 | 9.64 | 10.79 | 9.61 | 41.66 | 9.30 | 10.34 | 9.11 | 4.95 | 5.45 | 6.12 | 4.60 |
| Deck axial load on piers | 33.40 | 22.71 | 27.65 | 26.73 | 88.75 | 23.16 | 28.35 | 27.34 | 17.17 | 11.78 | 15.59 | 13.90 |
| Axial load at top of column | 19.07 | 16.31 | 18.61 | 16.50 | 23.29 | 13.01 | 20.74 | 18.58 | 14.25 | 4.56 | 7.56 | 5.70 |
| Vertical deflection at mid-span | 130.1 | 126.9 | 128.7 | 128.3 | 187.4 | 144.6 | 146.3 | 145.9 | 67.26 | 61.50 | 63.58 | 63.23 |



Table 5. Variation of Loads in Continuous Model Because of VCE From R.S Analysis

| | Tabas | | | | Kobe | | | | Northridge | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | TH | SRSS | 100+40 | 100+30 | TH | SRSS | 100+40 | 100+30 | TH | SRSS | 100+4<br>0 | 100+30 |
| Moment of deck at mid-span | 13.14 | 13 | 13.60 | 13.42 | 3.69 | 30.70 | 31.20 | 31.10 | 28.70 | 27 | 27.50 | 27.30 |
| Moment of deck on piers | 14.88 | 3 | 9.52 | 7.15 | 22.30 | 20.30 | 20.50 | 15.40 | 38.80 | 18.50 | 18.13 | 13.50 |
| Deck shear on pier | 17.30 | 7.70 | 8 | 6 | 64.45 | 30.60 | 33.80 | 32.60 | 30.30 | 25.80 | 28.90 | 27.60 |
| Pier axial force | 6.55 | 2.34 | 6.29 | 5.32 | 5.70 | 9.33 | 15.70 | 13.10 | 7.30 | 8.350 | 14.30 | 11.90 |
| Vertical deflection at mid-span | 67.26 | 61.50 | 63.50 | 63.20 | 187 | 144 | 146 | 146 | 130 | 127 | 128.70 | 128.40 |

Table 5 shows the results of second model from R.S analysis. The results show that VCE can increase the deck shear about 30% on piers and the moment about 20% at mid-span. Also, VCE can cause 10% variation on pier axial force and 130% increasing on superstructure vertical deflection.

## 8. Conclusion

The purpose of this study was to investigate the effect of VCE on continuous and monolithic bridges. To this end, 2 models were made and linear response spectra and nonlinear time history analysis were performed on models.

The results of T.H and R.S analysis on Monolithic bridge show that VCE can increase or decrease the axial force of piers and deck about 40%. The 40% variation can happen in shear force and moment of deck, too. However, VCE does not have any important effect on horizontal deflections.

The results show that the effect of VCE on continuous bridges are less than monolithic bridges. The results show that VCE can increase the superstructure moment and shear about 20% and 30%, respectively. The results show a 10% increasing in axial force piers.

Finally, it is recommended to enter the load factor of 0.6-1.4 DL in monolithic bridge design and 0.8-1.2 DL in continuous bridge design to consider the VCE effect on these kinds of bridges.

## References

[1] AASHTO. (1996). "Standard specifications for highway bridges, 16th ED." Washington. D.C

[2] M.Ala. Saadeghvaziri and D. A. Foutch, (1991) "Nonlinear Response of RC Highway Bridges Under the Combined Effect of Horizontal and Vertical Earthquake Motion" Journal of Earthquake Engineering and Structural Dynamics. Vol. 20, No 6, June 1991

[3] Broekhuizen. D. S (1996) "The Effect of Vertical Acceleration on Prestressed Concrete Bridges" MS thesis, Univ. of Texas at Austin, Tex.

[4] Yu. C. P., Broekhuuizen. D. S., Roesset, J. M., Breen, J. E., and Kreger, M. E. (1997) "The Effect of vertical ground motion on bridge deck response"Proc Workshop on Earthquake Engineering Frontiers in Transportation Facilities, Tech. Rep. No. NCEER-97-0005, National Center of Earthquake Engineering Research, State Univ. of New York at Buffalo. N.Y., 249-263

[5] Elnashai A. S, (1996) " Analysis and field evidence of the damping effect of vertical earthquake ground motion " Journal of Earthquake Engineering and Structural Dynamics, VOL. 25

[6] Yan Xiao, AsadEsmaeily, (1999) "The Effect of Variable Axial Load on Seismic Behavior of Bridge Piers "USC Structural Engineering Research Report, No. USC-SERP 99/01

[7] Martin R. Button, PE. , M.ASCE ; Colman J. Cronin ; and Ronald L. Mayes, M.ASCE, (2002) "The Effect of Vertical Motion on Seismic Response of Highway Bridges" Journal of Structural Engineering , Vol. 128, No 12

[8] Silva W. J. (1997) "Characteristic of vertical ground motions for application to engineering design" Proc., FHWA/NCEER Workshop on the National Representation of Seismic Ground Motion for New and Existing Highway Facilities, Tech. Rep. No. NCEER-97-0010, National Center for Earthquake Engineering Research, State Univ. of New York at Buffalo, N.Y, 205-252

[9] " Road and Railway Bridges Seismic Resistant Design Code" (2008), Ministry of Roads and Transportation Deputy of Training of Islamic Republic of Iran; Research and Information Technology, Publication No.463

[10] "Seismic resistant design of buildings – Code of practice",(2008) Institute Of Standards And Industrial Research Of IRAN, (2008)