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EARTHQUAKE RESISTANCE OF A MULTILEVEL INTERCHANGE BRIDGE SUPPORTED BY A SINGLE COLUMN

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ABSTRACT

The study tries to investigate the earthquake resistance of a five-level interchange bridge. Three dynamic analytical models were adopted for various purposes. Two aseismic design levels P1 and P2 were presented according to the earthquake risk analysis. Strength and ductility of the structure were estimated. The results indicate that the interchange bridge is safe from the fortuity earthquake but the great lateral displacement at the top of the highest pier would happen under the action of rare earthquake. The damping measures should be taken.

KEYWORDS

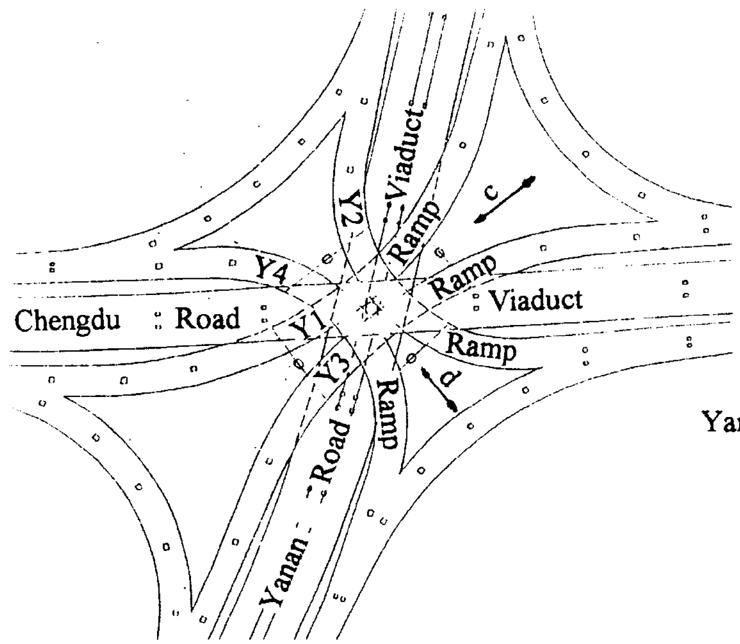
Multilevel interchange bridge; nonlinear earthquake response; coupled vibration; strength check; ductility analysis

INTRODUCTION

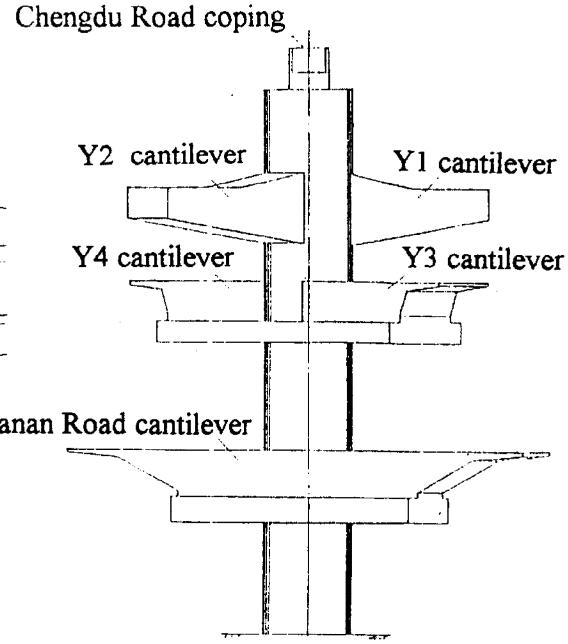
In Recent years, earthquake happened frequently in the world. It caused the collapse of many highway bridges, viaducts and freeways. So structure aseismic design becomes very significant.

In Shanghai, a five-level interchange bridge supported by a single column, which is located at the crossing of Chengdu Road Viaduct and Yanan Road Viaduct, has been constructed. The first level is ground road. The second level is Yanan Road Viaduct. The third level, Y1 and Y3 ramp, and the fourth level, Y2 and Y4 ramp, are left-turn ramps. The fifth level, which is the highest level, is Chengdu Road Viaduct. The simple deck of each viaduct is supported by a cantilever girder, which is cantilevered from a large central column. The diameter of this column is 4.2 m. The elevation of the highest deck is 34.6m. (see Fig.1) The five levels are coupled by the strong central column, expansion joints and rubber bearings. For such an interchange, the aseismic analysis becomes complex because of coupled vibration of the structure and the randomness of spread directions of seismic waves.

The study aims to present an analytical method, which can be applied to analyze special viaduct structure.



Plan view of the interchange



The center column

Fig.1

DYNAMIC ANALYTICAL MODELS

Three dynamic analytical models were adopted. Each model was composed of four levels of simple deck, and in each level, two spans which were arranged at the both sides of the central column were included.

Model I: The bottoms of the central column and all side piers were fixed. The deck was hinged to the piers in longitudinal direction and vibrated together with the piers in lateral direction. (see Fig.2)

Model II: The boundary conditions were same as model I except adding the elements of rubber bearings and expansion joints. According to their hysteresis loops, they were taken as elastic elements. (see Fig.2)

Model III: The pile foundation was considered based on model II. The spring elements were set on the nodes of piles in the longitudinal and lateral directions to simulate the resistance of soil. (see Fig.3)

Model I was the simplest one. It was used to determine the most dangerous input direction of seismic waves and to determine the critical sections using the response spectrum method. Model III was the most conformable one with the practical structure but it took a great deal of computer time. Calculation results showed that, the difference of inner force response between model II and model III was not over 20% , so a large number of computation cases of linear and nonlinear earthquake response analysis using time-history analysis method were completed on model II, and model III was used to check.

In displacement analysis of model II, the substructural method was used to consider the effect of following spans of Chengdu Road Viaduct and Yanan Road Viaduct. As shown in Fig.4, an equivalent element was used to replace the succeeding structure. Its axial stiffness is equal to the whole thrust stiffness of following n-span. The element joins up the pier and the deck with a rubber bearing and an expansion joint respectively. The analysis of dynamic behavior showed that the first twenty frequencies remain almost stable when $n \geq 12$, which means that the succeeding structure after the twelfth span has little effect on the dynamic behavior of the interchange. Therefore, twelve following spans were considered.

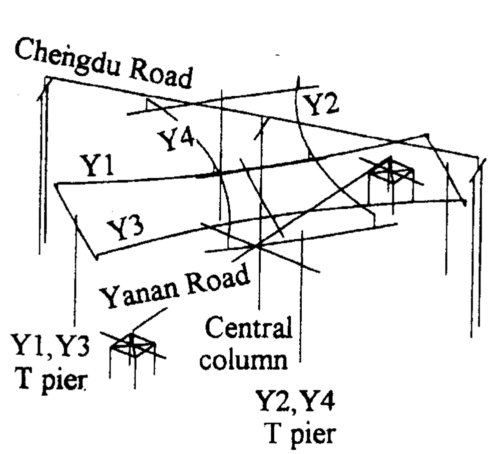


Fig.2 analytical model I, II

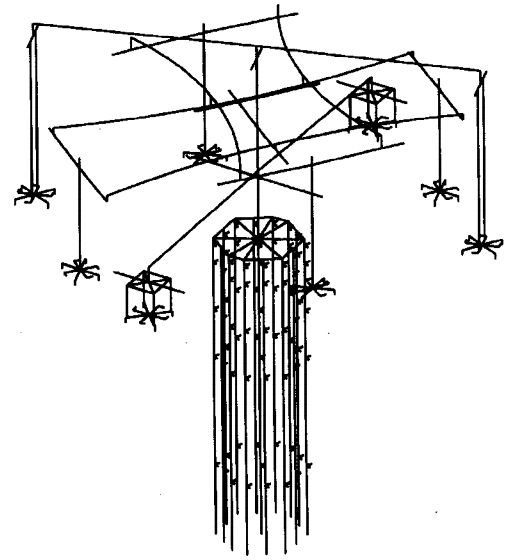


Fig.3 analytical model III

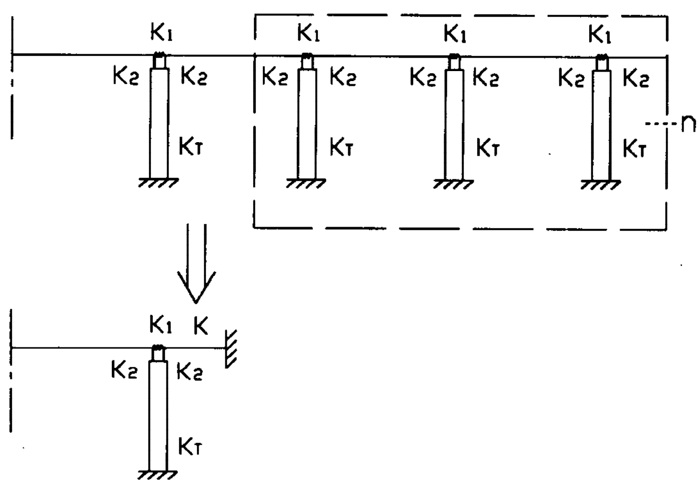


Fig.4 the simplification of succeeding structure

INPUT MOTIONS

Input earthquake motions

According to Chinese aseismic design code for bridges and buildings, three categories of structure performance standard were set: under the frequent earthquake, the structure should perform elasticity without damage; under the fortuity earthquake with 0.1 probability of exceedance in 50 years (P1), the structure may suffer repairable damage, but it should keep the emergency passage; under the rare earthquake with 0.1 probability of exceedance in 100 years (P2), the structure may suffer serious damage, but not collapse. In general, P1 is used to control strength and P2 is used to control displacement.

The input ground motions corresponding to P1 and P2 were presented based on the earthquake risk analysis.

The peak values of artificial ground acceleration records with two risk levels were listed in table 1.

Table 1 Peak value of artificial ground acceleration records (gal)

Direction	Risk Level P1	P2
Horizontal	156.875	190.480
Vertical	121.003	148.645

The determination of input direction

Owing to the space models and the randomness of spread directions of seismic waves, the most dangerous input direction must be determined.

Because Chengdu Road and Yanan Road , Y1,Y3 ramps and Y2,Y4 ramps almost cross at a right angle respectively, the following four input directions were selected: a. along Chengdu Road; b. along Yanan Road; c. along Y1,Y3 ramps; d. along Y2,Y4 ramps. (see Fig.1) Then using response spectrum method, the maximum inner forces response corresponding to different input directions can be obtained. By comparison, the most dangerous input directions and relevant control sections can be selected. They are listed in table 2. It can be seen that a and b are the most dangerous directions.

Table 2 Control sections and their most dangerous input directions

Control section	The most dangerous input direction
Bottom of the central column	a
Bottom of Chengdu Road pier	b
Bottom of Y1,Y3 T pier	a
Bottom of Y2,Y4 T pier	b
Bottom of Yanan Road pier	b
Fixed-end of cantilevers on Y1,Y3 T piers	a
Fixed-end of cantilevers on Y2,Y4 T piers	b
Fixed-end of Y1,Y3 cantilevers on the central column	b
Fixed-end of Y1,Y3 cantilevers on the central column	a

On the following sections, linear and nonlinear seismic response analysis of model II were executed using time-history method. Horizontal seismic waves were input along a and b direction respectively. Vertical seismic component which was taken as 2/3 horizontal component was input simultaneously. The results that were used to exam the strength of the structure can be found by combining seismic response with static inner forces.

LINEAR SEISMIC RESPONSE ANALYSIS

The response of model II under the action of the artificial seismic waves with risk level P1, is analyzed by means of time-history analysis method. The strength of each pier is checked against its yield surface, and the strength of each cantilever girder was checked against its resistance capacity.

Strength check of piers

Yield surface Yield surface, which indicates the interaction of yield strength P_u, M_{2u} and M_{3u} , can be expressed as the following equation, which was presented by Breseler:

$$\left|\frac{M_{2u}}{M_{2P}}\right|^a + \left|\frac{M_{3u}}{M_{3P}}\right|^b = 1 \quad , \quad \text{provided Pu is a constant.}$$

In which: a, b are dependent on section shape, generally speaking, for oval cross sections and rectangle sections, a=b=2 ; M_{2p} is yield bending moment of Pu about the principal Axis 2 (lateral), let $M_{3u}=0$; M_{3p} is yield bending moment of Pu about the principal Axis 3 (longitudinal), let $M_{2u}=0$.

M_{2p} and M_{3p} can be approximately expressed as:

$$\left|\frac{M_{2P}}{M_{22}}\right| = 1.0 + a_1 \cdot \left(\frac{P_u}{P_0}\right) + a_2 \cdot \left(\frac{P_u}{P_0}\right)^2 + a_3 \cdot \left(\frac{P_u}{P_0}\right)^3$$

$$\left|\frac{M_{3P}}{M_{33}}\right| = 1.0 + b_1 \cdot \left(\frac{P_u}{P_0}\right) + b_2 \cdot \left(\frac{P_u}{P_0}\right)^2 + b_3 \cdot \left(\frac{P_u}{P_0}\right)^3$$

$$-P_0 < P_u < P_t$$

where, P_t is single-axis yield tension; P_0 is single-axis yield pressure; M_{22} is pure bending yield moment about axis 2; M_{33} is pure bending yield moment about axis 3; $a_1, a_2, a_3, b_1, b_2, b_3$ are constants.

P_t, P_0, M_{22}, M_{33} are control points of yield surface for two principal axes, and when the values of these points are found, such parameters as $a_1, a_2, a_3, b_1, b_2, b_3$ can be easily calculated by above equations.

Strength check against yield surface The strength checks of piers can be easily carried out after their yield surfaces are obtained. Here, the central column is taken for an instance.

Since the section of central column is a circle (see Fig.5), the strength is constant in any direction, and the resultant bending moment Mu of bending moment about two principal axes M_{2u}, M_{3u} can be obtained according to vector composition rule, the yield surface of central column section can be simplified as the P-M curve shown in Fig.6.

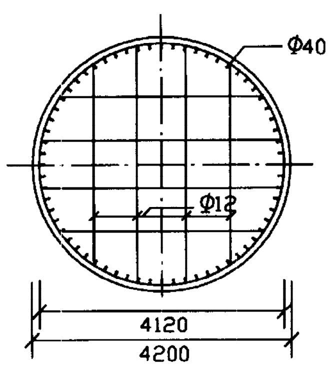


Fig.5 bottom section of central column

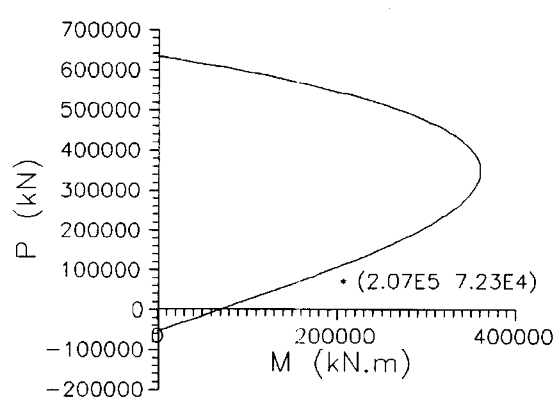


Fig.6 yield surface of central column

Fig.6 shows that the point (Pu,Mu) falls outside of the yield surface, so the column yield.

All other piers were analyzed in the same way, results show that all piers but Yanan Road piers in the interchange yield and work in a plastic state. Therefore, it is essential to analyze the nonlinear seismic response of the structure, and the ductility of each yield pier should be checked.

Strength check of cantilever girders

Since most piers yield under the action of artificial seismic waves with risk level P1, the nonlinear time-history response of model II was analyzed. In the analysis, yield piers were taken as nonlinear elements , while long cantilever girders were taken as linear elements. The results are listed in table 3. It can be seen that, for Y1,Y3 and Y2,Y4 cantilever girders on the central column, the seismic inner forces at the sections of fixed end control their design, but they are less than their resistance capacity respectively.

table 3 The maximum inner forces of cantilever beams

Control section (fixed-end)	Static analysis		Seismic analysis		Resistance force
	Dead load	Combined load	Static load + seismic load		
	M_{2g}(kN.m)	M_2^0(kN.m)	M_2(kN.m)	M_3(kN.m)	M_{2P}(kN.m)
Cantilevers on Y2,Y4 T piers	5.402×10^4	8.052×10^4	6.029×10^4	4.94×10^3	----
Cantilevers on Y1,Y3 T piers	5266×10^4	7.879×10^4	5.570×10^4	9.62×10^3	----
Y2,Y4 cantilevers on the central column	3.000×10^4	4.427×10^4	4.306×10^4	5.31×10^3	7.786×10^4
Y1,Y3 cantilevers on the central column	3.112×10^4	4.748×10^4	6.974×10^4	9.09×10^3	7.649×10^4

In which: direction 2 ---- longitudinal ; 3 ----vertical

NONLINEAR ASEISMIC RESPONSE ANALYSIS

Analysis of displacement at the top of tall piers

In asesmic analysis, the displacement is often used to estimate the safety of the structure under strong shock. Therefore, the displacements at the top of tall piers were emphatically analyzed, when artificial waves with risk level P2 were input. The maximum displacements of four analytical models at the top of tall piers are given in table 4. In the table, model A is just model II; model B, considering the effect of retaining block only; model C, considering the effect of succeeding structure only; model D, considering both effects. Among the four models, model D is the closest one to the normal state of the structure, model C is the next, while model A and model B will also exist if the first succeeding span falls. The table shows that, if the effect of retaining block or succeeding structure is considered, longitudinal displacements at the top of high piers decrease considerably, while lateral displacements change not dramatically.

Table 4 maximum displacements at the top of tall piers (cm)

Site	Analytical model	Longitudinal input along Chengdu Road: Longitudinal displacement	Longitudinal input along Chengdu Road: Lateral displacement	Longitudinal input along Yanan Road: Longitudinal displacement	Longitudinal input along Yanan Road: Lateral displacement
Pm108	A	22.10	15.63	9.16	38.85
	B	15.37	9.01	9.38	27.42
	C	10.93	2.64	4.93	40.23
	D	9.65	2.53	6.10	30.00
The central column	A	15.85	5.30	6.65	7.86
	B	12.91	5.17	7.15	8.68
	C	4.42	1.98	2.36	6.32
	D	6.94	5.72	3.99	8.15
Pm110	A	74.97	14.86	27.90	49.07
	B	17.73	11.00	11.20	42.91
	C	11.09	3.85	3.63	55.45
	D	11.23	2.97	6.23	42.92

Ductility analysis

According to the structure performance standard under earthquake, when the structures perform in plastic state under strong shock, the ductility can be used to judge the possibility of collapse. The ductility of a section can be described in its bending moment-curvature curve. In the paper, bending moment-curvature curves of sections of all piers were analyzed considering the restraint action of stirrups, then their ultimate curvatures and yield curvatures were determined. Thus, the ultimate plastic angle θ_p of each section can be calculated by the following formula:

$$\theta_p = \left(\varphi_u - \varphi_y\right) \cdot l_p$$

in which, φ_u, φ_y are ultimate curvature and yield curvature of the section respectively; $l_p = \frac{2}{3}D$ (D is depth of the section) is the length of plastic zone, according to European Code.

Table 5 lists the maximum plastic angles of each section at the bottom of piers in model II, which is under the action of artificial waves with risk level P2. Based on the plastic angles, the safety of sections can be valued. The table shows that , the maximum plastic angle of each section is smaller than its ultimate plastic angle.

Table 5 maximum plastic angles of piers (rad)

Control section (at the bottom of)	plastic angle θ_2	Ultimate plastic angle θ_{2p}	plastic angle θ_3	Ultimate plastic angle θ_{3p}
The central column	2.274×10^{-4}	1.68×10^{-2}	4.068×10^{-4}	1.68×10^{-2}
Chengdu Road piers	1.189×10^{-2}	2.51×10^{-2}	8.659×10^{-3}	2.65×10^{-2}
Y2,Y4 T piers	4.442×10^{-4}	8.65×10^{-2}	3.211×10^{-4}	8.53×10^{-2}
Y1,Y3 T piers	2.189×10^{-4}	8.65×10^{-2}	4.567×10^{-4}	8.53×10^{-2}

In which: direction 2 ---- lateral ; 3 ---- longitudinal

CONCLUSION

The following conclusions can be obtained based on the foregoing analysis:

1) Suffering from artificial seismic waves with risk level P1, piers in the interchange except Yanan Road piers yield, and work in plastic state. Further, nonlinear time-history response of the structure should be analyzed, and the ductility should be used to judge the possibility of collapse.

2) The critical sections of long cantilever girders are at the fixed end. Seismic inner forces of Y2,Y4 T-pier cantilevers and Y1,Y3 T-pier cantilevers do not control their design, while seismic inner forces of Y1,Y3 cantilevers and Y2,Y4 cantilevers on the central column control their design.

3) Suffering from artificial seismic waves with risk level P2, displacements at the top of Chengdu Road tall piers are very large. If neither the effect of retaining blocks nor the effect of succeeding structure is considered, the maximum longitudinal displacement reaches to 75 cm, and the maximum lateral displacement is 49 cm; while if both effect are considered, then the maximum longitudinal displacement is 11.23 cm, and the maximum lateral displacement is 42.49cm. Therefore, the lateral vibration control apparatus is suggested to be set up. The results show that, much attention should be paid to the aseismic design of tall piers in interchange bridges.

4) Suffering from artificial seismic waves with risk level P2, the maximum plastic angle at the bottom of each pier is smaller than its ultimate plastic angle, and there is no collapse possibility.

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