SEISMIC RETROFIT OF LONG BRIDGES IN THE HANSHIN EXPRESSWAY

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SUMMARY

After the Hyogo-ken Nanbu earthquake, the new specifications has been applied to bridges in Japan. For short and medium bridges, standardized methods can be applied to satisfy the new standards by improving ductility. On the other hand, there is no definite retrofit method for large bridges so far. In the Osaka Bay Route of the Hanshin Expressway, there are several types of long span bridges, and it has become imperative to find the suitable method to retrofit them so that the new seismic design standards can be satisfied.

For this purpose, a committee named "Evaluation on Seismic Stability of Long Bridges in Use" was formed, and the problem of seismic design, the result of analysis and the method of seismic retrofit were discussed for each long bridge. In this committee, guidelines were also drafted for the purpose of unifying the basic idea and the level of retrofit. The assumed seismic force, the permissible limit state, the method of response analysis, assessment and seismic retrofit were studied according to the guidelines, and possible retrofit methods for each long bridge were proposed.

INTRODUCTION

After the 1995 Hyogo-ken Nanbu earthquake, the new specifications has been applied to bridges in Japan. For short and medium bridges, standardized methods can be applied to satisfy the new standards by improving ductility. On the other hand, retrofit of large bridges is quite difficult because of their size and special features, and effective and realistic method must be considered for each case. In the Hanshin Expressway system, and especially in the Osaka Bay Route, there are several types of long span bridges such as cable-stayed bridges, truss bridges, and Neilsen-Lohse arch bridges. It has become imperative to find the suitable method to retrofit them so that the new seismic design standards can be satisfied. For this purpose, a committee named "Evaluation on Seismic Stability of Long Bridges in Use" was formed, and the problem of seismic design, the result of analysis and the method of seismic retrofit were discussed in the subcommittee for each long bridge.

In this committee, firstly the previous seismic design data were investigated, and the policy of seismic retrofit was discussed. Then, seismic responses were analyzed for possible strong earthquake inputs considering nonlinear property. By evaluating the responses, possible methods were proposed for each bridge, and finally suitable one was recommended, considering reliability, feasibility and cost-performance. For the purpose of unifying the basic idea and the level of retrofit of existing long bridges, "Seismic Retrofit Guidelines on Existing Long Bridges" was drafted. In the guidelines, the basic idea of assumed seismic force, permissible limit state, response analysis, assessment and seismic retrofit are specified as the result of discussion, advice and recommendation in the committee. The committee was started in 1997 and continued until 1998 when the basic study was completed. After receiving the proposal, the detailed design and retrofit work of each bridge are to commence.

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In this paper, seismic retrofit of long bridges is introduced mainly according to the working results of the committee, where the dynamic stability during strong earthquake motion and the policy of seismic retrofit were discussed, and possible retrofit methods for each long bridge were proposed.

RETROFIT ACCORDING TO DISCUSSION AND ADVICE OF THE COMMITTEE

Lessons learned from the Hyogo-ken Nanbu earthquake

After the 1995 Hyogo-ken Nanbu earthquake, some important knowledge can be obtained with respect to long bridges in the Osaka Bay Route. Main damage of long bridges occurred in supports (bearings, pendulum shoes) and seismic restrainers, and the displacement of a superstructure was very large reflecting the size of a structure [Ishizaki and Sawanobori, 1996]. Some structures had local buckling which was not fatal and could be repaired [Yonekura and Shimura, 1996]. From the experience of the earthquake, the following may be deduced for long bridges in the Osaka Bay area: [1] Most damage of a main structure was medium and repairable, but damage of bearings and other supporting system may cause fatal damage in result. Fatal damage can be avoided if suitable restrainers are installed. For structurally important members, failure is not permissible, while failure may be permissible for other secondary members according to the situation. [2] Each bridge has its own damage pattern, and the patterns may be classified by the type of a bridge. [3] Structural analysis can follow the response during an earthquake fairly well provided that supporting conditions of analysis is consistent with the fact and that suitable stress-strain relation is applied to the analysis [Ishizaki, Kitazawa, Nishimori and Noguchi, 1998].

General policy for seismic retrofit

In determining the retrofit method of long bridges in the Hanshin Expressway, the following is considered to unify the basic idea as illustrated in figure 1.

Input seismic motion

Basically, the seismic load specified in the “Specifications for Highway Bridges (1996)” shall be used as a seismic design load, and the most unfavorable one of either type I or type II spectrum-compatible accelerograms can be used in analysis, where the intensity of the type I or type II earthquake corresponds to the Kanto earthquake (1923) or the Hyogo-ken Nanbu earthquake (1995) respectively. It is also desirable to use a specific artificial earthquake motion estimated by a fault simulation model additionally if an active fault is situated near the bridge site.

Permissible damage state

Seismic damage must be limited to the level that a bridge can be re-used by repair or reinforcement. Hazardous situation (i.e. collapse or falling down of a bridge) must de avoided in the first place, and then damage should be as minor as possible, aiming at the state that emergency cars can pass the bridge just after an earthquake.

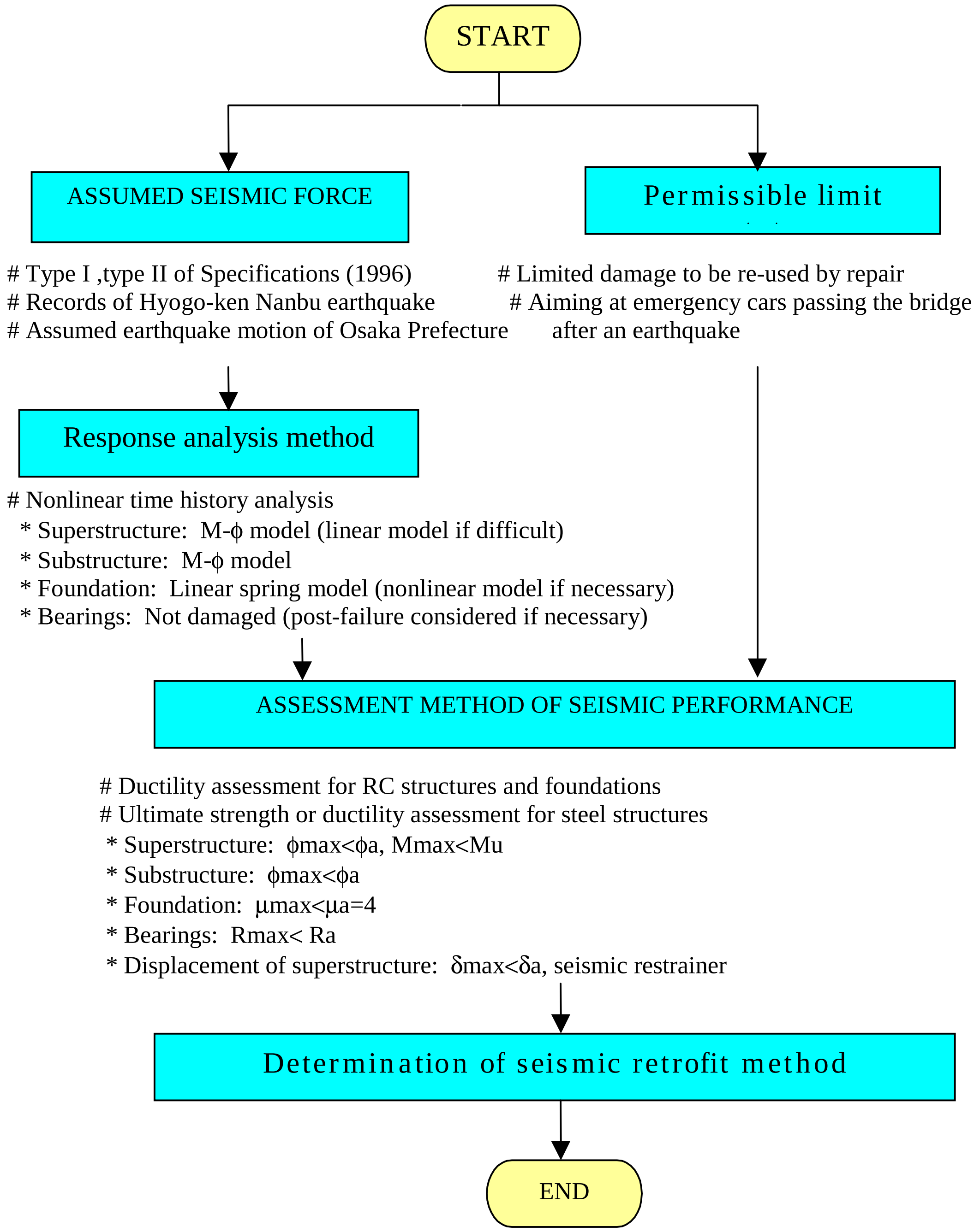

Figure 1: Flow of seismic retrofit procedure

Response analysis method

Basically, nonlinear time history analysis shall be used for a response analysis method so as to evaluate a complicated behavior of long bridges. The analytical model including the influence of a superstructure, a substructure and foundation-soil interaction shall be applied to dynamic analysis, and 3-D model may be also used if necessary. For simplicity of procedure, the next two steps can be taken for a superstructure: Step-1: As preliminary analysis, linear dynamic analysis may be applied, and members the stress of which exceeds a yield point can be picked up. Step-2: Necessary nonlinear property is taken into dynamic analysis, considering the properties of yielded members and expected seismic performance. Bearings are usually assumed not to be damaged since the structural behavior after the failure of bearings is uncertain, and this assumption is considered to be the safe side. However, in evaluating the post-failure behavior of bearings, the properties after the failure should be taken into the analysis.

Assessment method of seismic performance

(1) Superstructures (Steel structures)
Basically, seismic performance is assessed by the comparison of ductility capacity with a response ductility factor for type I or type II earthquake inputs. Yielding of some members is permissible to some extent, and safety is evaluated for each member to have a necessary safety margin against the ultimate strength. It is desirable to grasp the ultimate state of the whole structure in advance of the ductility assessment of each member, especially for statically indeterminate structures. If some of members does not satisfy the checking criteria, suitable measures should be taken such as improvement of ductility or strength, installation of collapse preventive devices, dampers, menshin bearings, and fail-safe devices to secure the required safety, and then the seismic performance is assessed again to assure the safety.

There are several cases in checking criteria according to the function of each member. Figure 2 illustrates a flow of assessment procedure for steel bridge members, and check level points and the corresponding assumed damage state are shown in figure 3.

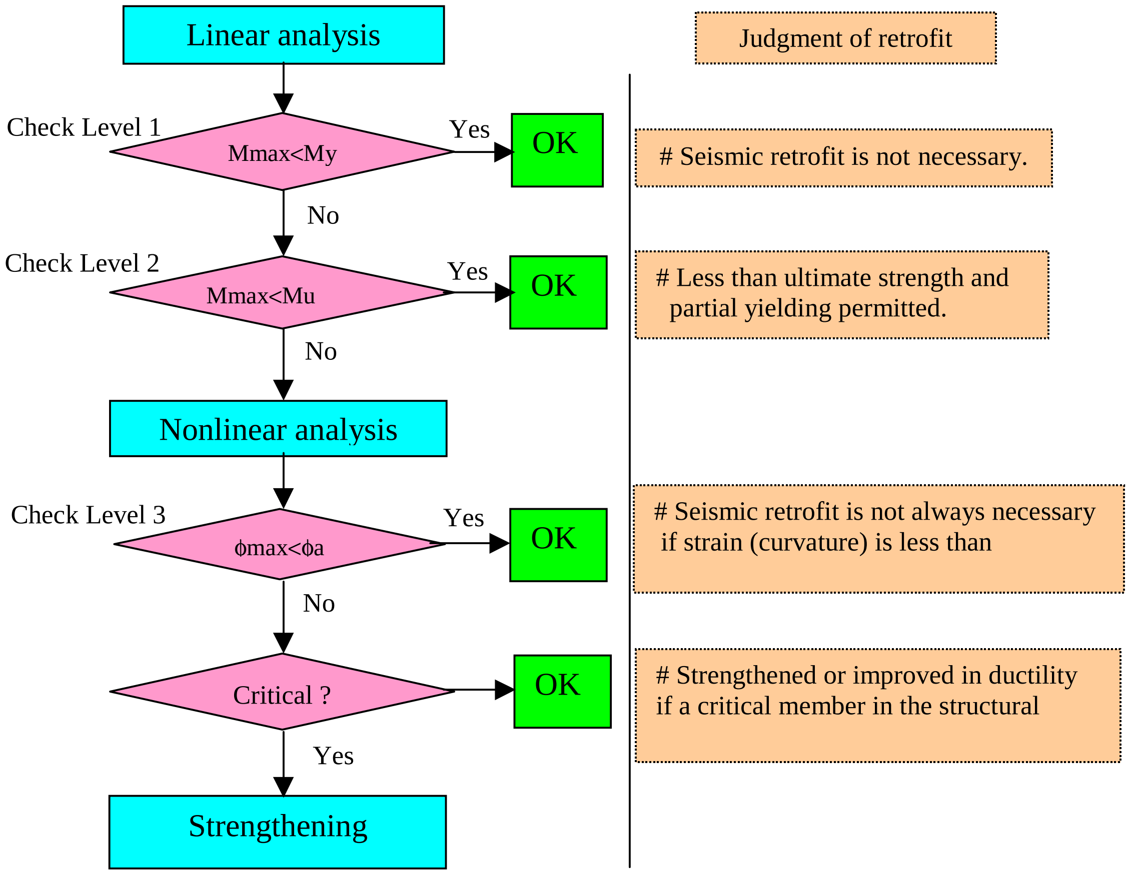

Figure 2: Flow of assessment procedure for steel bridge members

[A] Members subjected to bending moment exclusively (main girders etc.)
Generally, check level 3 can be omitted unless the whole bridge system becomes unstable by plastic hinges. Nonlinear model analysis is desirable to evaluate the influence of a nonlinear behavior to the dynamic response of the bridge system.
[B] Members subjected to bending moment and axial force (main tower, arch rib, etc.)
Basically, check level 3 can be applied in which a response ductility factor is compared with an allowable ductility factor. Generally speaking, a suitably designed steel column has enough ductility, and ductile design can be applied if the section is well controlled. An allowable ductility factor should be set suitable considering axial force and sectional control properties.

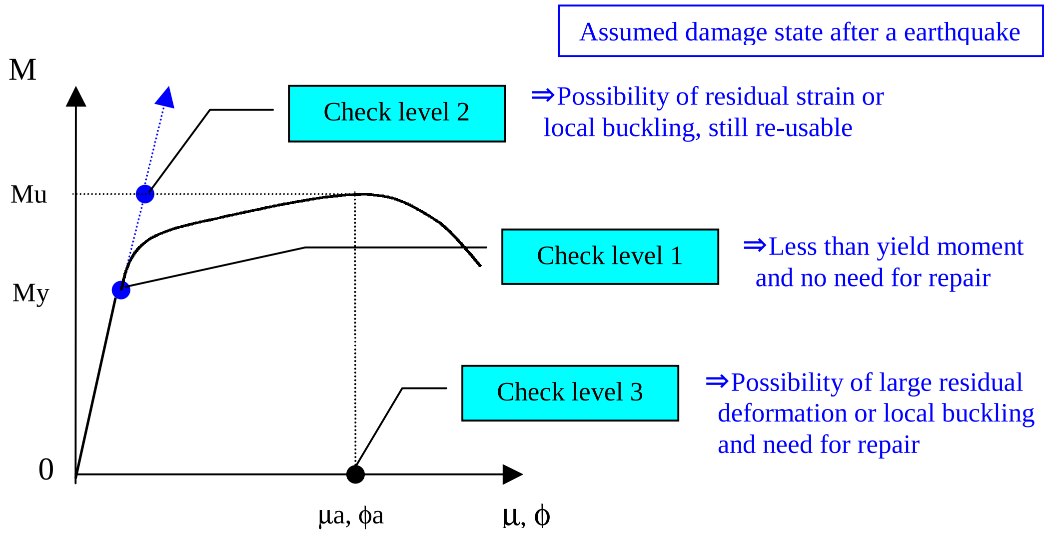

Figure 3: Each check level point for assessment

[C] <u>Members subjected to axial force (main truss etc.)</u>
Tensile strain of a member during an earthquake must be within allowable tensile strain which can exceed yield strain. On the other hand, compressive force of a member should be under the ultimate strength for buckling since the structural system become unstable, losing the bearing capacity after the buckling of compressive members. It is recommended to evaluate the post-buckling behavior of the whole system by elasto-plastic finite displacement analysis.

(2) <u>Superstructures (Concrete structures)</u>
For concrete bridges, the safety against type I or type II earthquake inputs shall be checked for the ultimate strength with yielding of axial reinforcement permitted. A response ductility factor (of curvature) calculated by nonlinear dynamic analysis shall be less than a allowable ductility factor of a member. Shear force by dynamic analysis must be less than the ultimate shear strength specified in the Specifications for Highway Bridges.

(3) <u>Substructures</u>
A reinforced concrete column with suitable hoops has usually enough ductility in bending, and a response ductility factor (of curvature) calculated by nonlinear dynamic analysis shall be less than a allowable ductility factor of a member. An allowable ductility factor should be set suitable considering materials and loading conditions of a member, where the ultimate curvature and other necessary coefficients can be calculated according to the Specifications for Highway Bridges. Shear force by dynamic analysis must be less than the ultimate shear strength specified in the Specifications for Highway Bridges, and the effect of an effective span ratio can be taken to the shear strength if the ratio is small enough. Steel piers are regarded as members subjected to bending moment and axial force, and safety can be checked in the same way as steel structures.

(4) <u>Foundations</u>
Judgment shall be made by comparing a response ductility factor of a foundation with an allowable ductility factor for type I or type II earthquake inputs. An allowable ductility factor should be set suitable considering the results of loading tests, past experience and site conditions.

(5) <u>Bearings and falling prevention system</u>
Bearings of existing long bridges shall be improved to resist seismic force together with seismic restrainers. Although the strength of bearings is not sufficient for type I or type II earthquake inputs in the case of long bridges, replacement of bearings is very difficult due to its size. Therefore, an idea that falling prevention system compensates bearings for lack of strength during an earthquake is realistic. The influence to adjacent girders should be also considered in the design of the falling prevention system since collision with a long bridge by excessive displacement may cause serious damage due to the difference of mass.

1.

EVALUATION OF SEISMIC RETROFIT FOR EACH LONG BRIDGE

Tenpozan Bridge

Outline of the bridge

Tenpozan Bridge is a three-span continuous steel cable-stayed bridge which is situated on the reclaimed land and crosses the mouth of the Aji River. The total length of the bridge is 640m with a center span of 350m, and the length of side spans are 170m and 120m reflecting the alignment and site conditions. The height of the girder is as high as 50m above the sea level to secure a clearance under the bridge. The supporting layer is soft and has a depth of 35m, and the design problems such as wind stability, earthquake resistant property and the construction method were fully investigated and studied. The cable in the superstructure is 2-plane fan pattern multi-cable system with 9 stay cables each plane. The main girder was designed to be a flat hexagonal box girder to improve wind stability. The main towers are A-shaped to improve the torsional rigidity, standing with flexibility on the foundation footings. The foundation consists of cast-in-place RC piles of 2m in diameter, and the bearing capacity was decided from the results of a loading test at the site. As to the aseismic design, the seismic force is expected to be dispersed by fixing the main girder at the both towers. The bridge is relatively flexible with a predominant period of 3.7seconds, and seismic force can be reduced. The behavior at the time of earthquakes is examined by dynamic analysis, using the acceleration response spectrum for design and considering dynamic interaction between foundations and soil.

The result of analysis and the seismic retrofit method

As a preliminary study, linear dynamic analysis was conducted, and it was found that there was a high risk of failure in bearings and that some part of towers would yield against the type I and type II earthquake. Taking the result of the preliminary study into consideration, nonlinear time history response analysis was conducted. In the analysis, elast-plastic model was taken into some possible members, and the towers were assumed to be fixed at the base. The type I and type II earthquake were used as input force respectively in the longitudinal and transverse direction of the bridge, considering the most unfavorable case. Table 1 shows the maximum response of displacement and force at the girder in the transverse direction, and table 2 shows the maximum displacement response of the girder in the longitudinal direction.

Table 1: The maximum response of displacement and force in the transverse direction

Max. value	Input earthquake	AP-1	AP-2	AP-3	AP-4
Response displacement (m)	Type I (on the average)	0.80	0.69	0.35	0.88
	Assumed motion of Osaka	0.61	0.35	0.28	0.74
Response force (tf)	Type II (on the average)	3,159	5,793	5,229	3,316
	Assumed motion of Osaka	2,893	4,910	4,891	3,256
Ultimate strength of bearings or wind shoes (tf)		1,497	4,080	4,502	1,497

Table 2: The maximum response of the girder in the longitudinal direction

Input earthquake	Maximum displacement (m)	Residual displacement (m)
Type -1	2.05	1.0
Type -2	1.98	0.5
Type -3	3.33	2.0
Average	2.46	1.2

The main results are as follows: [1] There are members whose stress exceeds a yield point, and the stiffeners with a large width-thickness ratio parameter should be reinforced. [2] A large horizontal force will act on the bearings, and there is a high risk of failure in bearings. Therefore, suitable displacement restriction system is necessary. [3] A large displacement occurs in the girder in the longitudinal direction, and falling prevention system is necessary. [4] It is judged that retrofit of the main girder, cables, base anchors and foundations is not necessary. Partial reinforcement for ductility improvement is to be done against the Type II earthquake since a part of the towers and piers goes into yielding. At the same time, the falling down prevention devices of tower bearings and the displacement restriction devices at the end piers are to be installed in order to secure the seismic safety.

Shinhamadera Bridge

Outline of the bridge

Shinhamadera Bridge is one of the longest Nielsen Lohse bridges in Japan, which has the center span of 254m. The bridge crosses the Hamadera canal between Sakai city and Senboku industrial complex on the reclaimed land. The type of main arch is so called a basket handle shape to improve the rigidity of the structure. The rise of the arch is 36m, and the rise ratio is 1/7, making an angle of arch inclination approximately 17 degrees. The stiffening girder and steel floor deck are designed composite from a economical standpoint. Cables are arranged every 12m with fixed inclined angle of 60 degrees, while the ends of cables are installed vertically to alleviate stress fluctuation. Nine upper lateral braces are placed as a result of examination on the appearance and buckling property. The ultimate capacity of the bridge was investigated on both horizontal and vertical loads by using the whole model respectively, and the elasto-plastic finite element analysis was also performed to ensure the stability. The piers are RC structures in a reversed trapezoidal shape from an aesthetic viewpoint, and prestress was introduced to the horizontal beam member. The foundation is cast-in place RC piles (ϕ2.0m) with a cap footing.

The result of analysis and the seismic retrofit method

As a preliminary study, linear response spectrum analysis was conducted for the type I and type II earthquake inputs. Then, the next several cases were studied to compare the effect of countermeasure; [1] time history response analysis considering the nonlinear property of RC piers, [2] time history response analysis considering the nonlinear property of RC piers and upper lateral braces, [3] time history response analysis considering the nonlinear property of RC piers and upper lateral braces reinforced by ribs, [4] linear response spectrum analysis where the steel bearings were replaced by rubber bearings. In the analysis, 3-D model was used, provided that bearings can function during an earthquake.

The main results are as follows: [1] The yielding of the arch will occur at the corner of upper lateral bracing for the transverse earthquake input. But there remains enough extra strength, and strengthening is not always necessary. [2] The strain of the upper lateral braces exceeds the ultimate strain, and the upper lateral braces need to be reinforced by ribs. [3] A large horizontal force will act on the bearings, and there is a high risk of failure in bearings. Therefore, suitable falling down prevention system is necessary. [4] Steel jacket reinforcement can be applied for RC piers to satisfy the necessary strength and ductility. [5] It is judged that retrofit of the main girder and cables is not necessary. As a retrofit method, the next was proposed: The upper lateral braces and the corner of them are to be reinforced by ribs or cover plates. Stoppers for bearings may be difficult to install, since the strength of the connection part of the substructure is insufficient. Seat widening should be used to increase the overlapped length of the girder at the support. RC piers can be retrofitted by steel jacket reinforcement method.

Minato Bridge

Outline of the bridge

Minato Bridge is a cantilever truss bridge which goes across the waterway in Osaka Port. Since a number of large ships are passing the waterway, the bridge was designed to have a long span of 510m and a high clearance, and becomes a symbol of Osaka Port with the third longest span in the world. In those days, cable-stayed bridges on such a large scale had not been constructed, and a cantilever truss was selected taking into consideration the land subsidence at the supporting points. Well-balanced K-truss was also adopted for the frame structure. In this bridge, high tensile strength steels (HT70, HT80) were used for top and bottom chords as much as 5000t. Such a large amount of high tensile strength steel had never been used in one bridge before, and good quality control and development were required in materials, production and welding. The bridge is so high and rigid that it is mainly influenced by wind and seismic load. For wind effect, wind tunnel tests were carried out to estimate the drag coefficient. As to seismic load, the modified seismic coefficient was adopted on the basis of dynamic analysis. The type of foundation in the intermediate supports is a large-scale pneumatic caisson, and bearing capacity was checked on the basis of the ultimate bearing capacity and consolidation yield strength of underlying diluvial clay layer. The central suspended simple girder with a span length of 186m was towed to the site by a barge and was lifted up to 60m above the sea in one lump, using winches placed on the cantilevered girders in both sides.

The result of analysis and the seismic retrofit method

As a preliminary study, linear dynamic analysis using 3-D model was conducted, and it was found that the force of truss members exceeded the allowable strength against the type I earthquake. In a truss bridge the ductility of the main structure can not be expected, and the nonlinear property of the foundation-soil was first introduced to the analysis. As a result, the force of truss members was still about two times as large as the allowable strength, and the displacement of the ground became 1.1m, though the response of the structure decrease by 20%-60%. Since it seemed difficult to find a suitable retrofitting method, the next several cases were studied and compared to seek for some clues to the countermeasure; [1] replacement of the steel bearings of the intermediate truss supports to rubber bearings, [2] installation of dampers, [3] base isolation of road slab and girders by rubber bearings, [4] application of type II earthquake inputs reflecting the local properties (or consideration of Uemachi fault in Osaka), [5] nonlinear dynamic analysis of truss members. In either case, necessary safety could not be satisfied, and even by the combined case, it was difficult to secure the safety. For Minato Bridge, strengthening of members on a large scale is difficult, and some measures to secure the safety of the whole system have to be considered. Since the force which exceeds the allowable maximum strength, works on the bearings, the countermeasure such as falling down prevention devices is firstly necessary. Then , a suitable retrofit method is to be selected by investigating the result in more detail, and at the same time, earthquake inputs considering the local properties, soil nonlinearity and the influence of large displacement to adjacent bridges are to be studied.

CONCLUSIONS

In this paper, seismic retrofit of long bridges was introduced mainly according to the working results of the committee, where the dynamic stability during strong earthquake motion and the policy of seismic retrofit were discussed. The main results are as follows:

[1] "Seismic Retrofit Guidelines on Existing Long Bridges" was drafted to unify the basic idea and the level of retrofit of existing long bridges. In the guidelines, the basic idea of assumed seismic force, permissible limit state, response analysis, assessment and seismic retrofit are specified as the result of discussion, advice and recommendation in the committee.

[2] The seismic stability and retrofit method were investigated for each type of long bridges, and possible methods were proposed for each bridge referring to the guidelines and advice of the committee.

[3] As for cable-stay bridges such as Tenpozan Bridge and Yamatogawa Bridge, seismic safety can be secured by partial reinforcement for ductility improvement in the tower, falling prevention system of tower bearings and displacement restriction at the end piers.

[4] Shinhamadera Bridge, which is a Nielsen type arch bridge, can also retrofitted by partial reinforcement at the corners of upper lateral bracing and falling prevention system of bearings. On the other hand, Minato Bridge, which is a cantilever truss bridge and the ductility of the main structure can not be expected, strengthening of members is difficult and becomes on a large scale, even if possible, and some measures to secure the safety of the whole system have to be considered.

[5] For long bridges with a substructure of RC pier type, the safety during an earthquake and the method of seismic retrofit can be studied by taking the nonlinear effect of a substructure into an analysis model, whereas for long bridges without RC piers, nonlinear analysis of the foundation-soil system is necessary to discuss the stability of the foundation and the effect on the superstructure if the ductility of the main structure can not be expected.

[6] The supporting conditions after the failure of bearings should be also investigated, and suitable countermeasures have to be studied by taking the supporting conditions into dynamic analysis. Earthquake inputs including the local properties and the influence of large displacement to adjacent bridges also remain to be studied.

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