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Modelling and Simulation of Reinforced Concrete Bridges with varying percentages of Shape Memory Alloy Rods

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ABSTRACT

Earthquakes constitute a major problem for mankind resulting in loss of lives and structures. Smart structural materials such as Shape Memory Alloy (SMA) suppress the structural vibration in a structure by adjusting the dynamic performance of the structure. SMA rods are unique for their shape memory effect and super elasticity and have been used as structural reinforcement for earthquake retrofits. This research focused on investigating the appropriate percentage of shape memory alloy and steel reinforcements for the least deflection in the column-capping beam of a 3-span composite Matsurube Bridge in Japan subjected to seismic load. Five different earthquake scenarios were used to obtain the best combination of steel and SMA reinforcement in the columns and capping beam for the best resistance to the earthquake response. Data used for simulations were obtained from the bridge components. It was observed that SMA has a high resistance to seismic loads when combined with steel reinforcement and it is therefore recommended for inclusion in reinforced concrete bridges to serve as means of reducing the effect of earthquakes on structures in earthquake prone areas.

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1. Introduction

In any seismic design and prevention plan, guaranteeing the resilience of bridges against earthquakes in densely populated areas is of high importance. Bridges are very important elements in transportation infrastructure network. Some studies carried out to identify the major cause of bridge collapse during seismic activities have shown that they are due to the collapse of one or more of the bridge reinforced concrete (RC) columns or piers [1]. This failure mode suggests that additional structural elements should be provided to complement the column resistance. [2] reported that Smart structural materials are gaining acceptance for use as different form of braces in concrete structures. The conventional method of mitigating seismic activities is to increase the stiffness of structural members by enlarging their section properties, this will accommodate the seismic load because of the added mass to structures. This technique brings about an increase in the cost of the structure while the safety level of such structures is little improved.

Another disadvantage of the conventional anti - seismic technique is that it focuses on protecting the structure at the expense of the facilities such as machines, fittings and finishes inside the structure. Hence, it cannot be used in some structures. Though it may be challenging to design a structure which is damage-proof to dynamic loads like earthquakes and strong winds, smart structural materials may help in reducing the vibration of structures [2]. Smart structural materials which are often installed or embedded in structures suppress the structural vibration in the structure by changing or adjusting the dynamic performance of the structure. Thus, the structures will be able to sense and respond to their surroundings in a desired and anticipated manner [3].

[4] defined smart materials as that which have the capability to modify their physical properties in a specified manner in response to precise stimulus input such as electric and magnetic fields, temperature, pressure, chemicals or nuclear radiation. The related modifiable physical properties are stiffness, shape, viscosity or damping. Smart structures have the capacity to sense and react to their environment in an expectable and desired manner through the incorporation of some elements such as actuators, power sources, sensors, signal processors and communications network. Smart structures can reduce vibration and acoustic noise, self-monitor their own conditions and surroundings, perform precision alignments automatically, and modify their mechanical properties and shape on command, in addition to carrying mechanical load.

Smart structural materials include shape memory alloy (SMA), piezoelectric materials, magneto-restrictive materials, tunable electromagnetic absorbers, photo-chromic windows and macro fiber composite [2]. Of all the Smart structural materials, Shape memory alloy performs most efficiently due to its shape memory effect and super elasticity. SMA recollects its original shape and returns to the pre-deformed shape when heated. It is a lightweight, solid-state substitute to orthodox actuators such as pneumatic, hydraulic, and motor-based systems. SMA are applied in medical and aerospace industries [5].

The aim of this research is to determine the appropriate percentage of SMA combined with steel reinforcements that will give the least displacement in a bridge's column and capping beam when subjected to seismic loads.

2. Shape memory alloys (SMA)

Nickel-titanium alloys possess superior thermos-mechanical and thermo-electrical properties making it to be more useful when compared to other SMAs [6]. Copper-zinc-aluminum, copper-aluminum-nickel, and iron-manganese-silicon alloys are other examples of SMA [7]. Nickel-titanium alloys are generally known as Nitinol. Nitinol SMAs have the ability to return back to their pre-set shapes when heated (shape memory effect) and the capability to undergo a large amount of inelastic deformation and recover their shape when loads are removed from them (super elasticity) [6]. These exceptional properties result from the reversible phase transformations of SMAs. Nitinol are manufactured in various forms such as rod and bar stock, wire, and thin film. The properties of Nitinol are shown in Table 1 and its properties are compared with typical structural steel in Table 2.

Table 1
Properties of binary Nitinol SMAs.

Property	**Value**
Melting temperature (ºC)	1300
Density (g/cm^3)	6.45
Resistivity of austenite (μΩcm)	≈ 100
Resistivity of martensite (μΩcm)	≈ 70
Thermal conductivity of austenite (W/(cmºC))	18
Thermal conductivity of martensite (W/(cmºC))	8.5
Corrosion resistance	similar to Ti alloys
Young's modulus of austenite (MPa)	≈ 80
Young's modulus of martensite (MPa)	≈ 20 to 40
Yield strength of austenite (MPa)	190 to 700
Yield strength of martensite (MPa)	70 to 140
Ultimate tensile strength (MPa)	≈ 900
Transformation temperature (ºC)	-200 to 110
Shape-memory strain (%)	8.5

Source: [8]

Table 2
Comparison of the mechanical properties of SMAs and Structural steel.

Property	NiTi SMA	Steel
Recoverable elongation (%)	8	2
Modulus of elasticity (MPa)	8.7x10^4 (A), 1.4x10^4 (M)	2.07x10^5
Yield strength (MPa)	200-700 (A), 70-140 (M)	248-517
Ultimate tensile strength (MPa)	900 (f.a.), 2000 (w.h.)	448-827
Elongation at failure (%)	25-50 (f.a.), 5-10 (w.h.)	20
Corrosion performance (-)	Excellent	Fair

Source: [8]
Where A = austenite, M = martensite f.a. = fully annealed and w.h. = work hardened

3. Methods

3.1. Description of the study area

The Matsurube Bridge is a three-span bridge constructed on National Highway Route #342 in Japan. It was constructed in 1978 but on June 14, 2008, the bridge collapsed because of the Iwate-Miyagi-Nairiku Earthquake with magnitude of 7.2. Figure 1 shows the map of the Iwate, Akita and Miyagi prefectures.

The superstructure of the Matsurube Bridge was made up of a 3-span continuous steel girder having a span of 94.9m. Each of the spans were 27m, 40m and 27m respectively. The superstructure was supported by two piers in the middle spans and two abutments at either ends. The width of the deck was 10m. The superstructure also consisted of 4 steel plate girders with concrete slab deck. Two piers in the middle spans were Reinforced Concrete wall type piers with a height of 25m, and both abutments were an inverted T-shaped Reinforced Concrete wall type. The ground condition was Type I (Stiff) according to the Japan Road Association (JRA) highway bridge design specifications and therefore all foundations of piers and abutments were a spread type. Seismic coefficient employed in the original design was 0.15 with allowable stress design method [9].

The three spans Bridge was simulated with 100% Steel reinforcement, 75% Steel reinforcement combined with 25% SMA, 50% Steel reinforcement combined with 50% SMA, 25% Steel reinforcement combined with 75% SMA and 100% SMA using Finite Element Method by SAP2000 Engineering software. The interface for the material Property definition for concrete where the weight per unit volume, the modulus of elasticity, Poisson’s ratio, coefficient of thermal expansion, shear modulus and the concrete strength were defined is shown in Figure 2. The interface for the material property definition for both 100% steel and 100% SMA reinforcements respectively, where the weight per unit volume of the reinforcements, modulus of elasticity, Poisson’s ratio, coefficient of thermal expansion, shear modulus yield and tensile stresses were defined are shown in Figure 3. The reinforcement property definitions were modified based on the percentage of steel and SMA reinforcements in the bridge. The interface for defining column and capping beam properties, both of which were modelled as frame sections, is shown in Figure 4. A typical structural detail of the Bridge with 50% steel reinforcement and 50% SMA is shown in Figure 5. The steel reinforcements are denoted with red colours while the SMA reinforcements are denoted with blue colours.

3.2. Numerical analysis for matsurube bridge

The finite element method (FEM) was used in simulating the bridge by assuming a nonlinear elastic behavior of the frame structure. The bridge deck, beams, capping beams and column frames were divided into members and nodes. Boundary conditions were applied based on the type of support conditions, the forces applied on the bridge and the displacements that were specified. The Horizontal elastic response spectrum from [10] and the ground type C response spectrum parameters, which corresponds to the Type I (stiff) soil in the Japan Road Association (JRA) highway bridge design specifications (describing a soil having large deposits of dense or medium-dense sand, gravel or stiff clay with large thickness) were used to determine the response of the Matsurube Bridge structure to ground acceleration. Five different earthquake accelerograms (Newhall, Elcentro, Santa-Monica, Petrolia and Pomona) were also used for the Time History analysis of the Bridge. The global stiffness matrix equations generated were solved and post processing of the results were obtained from SAP2000. Figure 6 shows an extruded view of the bridge model.

4. Results and discussion

The maximum reactions from the plate girders on the capping beam were recorded for the various load cases to which the bridge was subjected. Modal analysis was done to determine the periods of the first five natural modes of the bridge when reinforced with varying percentages of SMA under five different earthquake scenarios. The response spectrum for joints J1 and J2 as shown in Figure 7 (joint J1 is the joint between the pier and the base while joint J2 is the joint between the pier and the capping beam) were also plotted. The Spectral Accelerations (AA) was plotted against the period. The base moments, base shears and joint displacements at joints 1 and 2 of the Matsurube Bridge were also recorded.

The reactions from the four plate girders on the capping beam for the combination of load cases used (Dead load, Super Imposed Dead Load, Braking load, Wind load, and Moving Load) under Ultimate Limit State (ULS) as specified in Table 1 of [11] for which the bridge was analyzed is shown in Table 3.

From the load combinations, the maximum axial force is 4,505.41kN, the maximum vertical shear is 6,651.22kN, the maximum horizontal shear is 1,984.30kN, the maximum torsion is 1,671.44kN, the maximum vertical moment is 8,093.63kNm and the maximum horizontal moment is 29,336.22kNm.

Table 3
Reactions from the Plate Girders due to ULS load Combinations.

Reactions	Plate Girders			
	Left Exterior Girder	Interior Girder 1	Interior Girder 2	Right Exterior Girder
Axial Force (kN)	4505.41	4309.90	4166.02	4262.37
Vertical Shear (kN)	3229.75	6651.22	6443.28	2864.13
Horizontal Shear (kN)	1984.30	1984.26	977.34	1876.73
Torsion (kN)	986.70	1149.15	1671.44	683.46
Vertical Moment (kNm)	8093.63	612.55	661.32	1403.40
Horizontal Moment (kNm)	26512.26	29336.22	28727.21	24937.07

The natural periods for the first five natural modes vibration for the five earthquakes with varying percentages of SMA and Steel in the bridge columns are shown in Figures 8. The periods in seconds (s) of the first five modes for the five earthquakes which are presented in Table 4 were the same for the different percentages of SMA and steel. It was noticed that with the introduction of SMA the values of the natural periods began to reduce. The highest period is 0.536 seconds which occurred in the first mode when the bridge was reinforced with 100% Steel.

Table 4
Natural periods of the first five (5) natural modes of the Bridge for the Earthquakes.

100% Steel (s)	25% SMA + 75% Steel (s)	50% SMA + 50% Steel (s)	75% SMA + 25% Steel (s)	100% SMA (s)
0.536	0.504	0.486	0.474	0.465
0.325	0.274	0.251	0.239	0.231
0.291	0.266	0.249	0.236	0.229
0.261	0.242	0.232	0.226	0.221
0.241	0.218	0.200	0.184	0.170

The maximum spectral acceleration for joint J1 is 19.6 X 10^3g obtained from the Newhall earthquake while the maximum spectral acceleration for joint J2 is 34.4 X 10^3g also obtained from the Newhall earthquake. The response spectrum curve showing the spectral acceleration versus the Time period (seconds) for joints J1 and J2 for each of the earthquake scenarios for the 100% steel reinforced bridge is shown in Figures 9.

The Minimum and Maximum values for the base shears for the five earthquake scenarios are shown in Table 5. The maximum base shear is 1.445E+07kN under Newhall earthquake which occurred when the bridge was reinforced with 100% SMA. There was increase in the base shears with increase in the percentages of SMA.

Table 5
Minimum and Maximum Base Shears for Newhall Earthquake

Earthquakes		100% Steel	25% SMA	50% SMA	75% SMA	100% SMA
Newhall	Min (kN)	-4.546E+06	-5.222E+06	-7.560E+06	-9.816E+06	-1.256E+07
	Max (kN)	8.745E+06	9.564E+06	1.162E+07	1.330E+07	1.445E+07
Elcentro	Min (kN)	0	0	0	0	0
	Max (kN)	2.415E+05	2.380E+05	2.477E+05	2.450E+05	2.441E+05
Santa Monica	Min (kN)	-4.256E+06	-6.298E+06	-7.392E+06	-8.440E+06	-9.063E+06
	Max (kN)	8.735E+06	9.558E+06	1.162E+07	1.330E+07	1.444E+07
Petrolia	Min (kN)	-4.350E+06	-5.536E+06	-6.461E+06	-7.315E+06	-8.612E+06
	Max (kN)	8.645E+06	9.453E+06	1.157E+07	1.323E+07	1.437E+07
Pomona	Min (kN)	-2.537E+06	-3.941E+06	-4.611E+06	-5.384E+06	-6.225E+06
	Max (kN)	5.733E+06	6.269E+06	7.696E+06	8.801E+06	9.554E+06

The Minimum and Maximum values for the base moments for the five earthquake scenarios are shown in Table 6. The maximum base moment is 3.336E+07KNm under Newhall earthquake which occurred when the bridge was reinforced with 100% SMA. There was increase in the base moment with increase in the percentages of SMA.

Table 6
Minimum and Maximum Base Moments for Newhall Earthquake

Earthquakes		100% Steel	25% SMA	50% SMA	75% SMA	100% SMA
Newhall	Min (kNm)	-2.51E+07	-2.585E+07	-2.433E+07	-2.661E+07	-2.876E+07
	Max (kNm)	2.18E+07	2.612E+07	2.946E+07	3.185E07	3.336E+07
Elcentro	Min (kNm)	0	0	0	0	0
	Max (kNm)	1.02E+06	9.444E+05	9.034E+05	9.037E+05	9.033E+05
Santa Monica	Min (kNm)	-1.22E+07	-1.785E+07	-2.241E+07	-2.537E+07	-2.757E+07
	Max (kNm)	2.21E+07	2.557E+07	2.893E+07	3.133E+07	3.327E+07
Petrolia	Min (kNm)	-1.44E+07	-2.026E+07	-2.294E+07	-2.270E+07	-2.696E+07
	Max (kNm)	2.11E+07	2.491E+07	2.822E+07	3.058E+07	3.302E+07
Pomona	Min (kNm)	-8.94E+06	-1.326E+07	-1.226E+07	-1.350E+07	-1.754E+07
	Max (kNm)	1.46E+07	1.643E+07	1.847E+07	2.003E+07	2.189E+07

The Minimum and Maximum displacements at joints J1 and J2 for the five earthquake scenarios are shown in Table 7 and 8. There was no displacement at Joint J1 for all the earthquake simulations. It shows that the earthquake has no effect at this joint. The maximum displacement at Joint J2 is 8.375m under the Newhall earthquake when the bridge was reinforced with 100% Steel. It was noticed that the introduction of SMA brought about a reduction in the displacement at this joint by an average of 26.7%. The column displacement for the five earthquake scenarios during time history analysis were computed when the capping beam was reinforced with 100% Steel reinforcement and the columns were reinforced with 100% Steel reinforcement, 75% Steel combined with 25% SMA, 50% Steel combined with 50% SMA, 25% Steel combined with 75% SMA and 100% SMA. The results are plotted on a graph of Displacement (mm) against Distance of column from capping beam (m) and shown in Figure 10. The results revealed that the introduction of SMA in the columns reduced the maximum displacement in the columns by an average of 42.710% when reinforced with 75% steel and 25% SMA, 52.318% when reinforced with 50% steel and 50% SMA, 62.258% when reinforced with 25% steel and 75% SMA and 74.088% when reinforced with 100% SMA.

Table 7
Minimum and Maximum Displacements at Joint J1 Earthquake

Earthquake		100% Steel	25% SMA	50% SMA	75% SMA	100% SMA
Newhall	Min (m)	0	0	0	0	0
	Max (m)	0	0	0	0	0
Elcentro	Min (m)	0	0	0	0	0
	Max (m)	0	0	0	0	0
Santa Monica	Min (m)	0	0	0	0	0
	Max (m)	0	0	0	0	0
Petrolia	Min (m)	0	0	0	0	0
	Max (m)	0	0	0	0	0
Pomona	Min (m)	0	0	0	0	0
	Max (m)	0	0	0	0	0

Table 8
Minimum and Maximum Displacements at Joint J2

Earthquakes		100% Steel	25% SMA	50% SMA	75% SMA	100% SMA
Newhall	Min (m)	-9.347E+00	-7.881E+00	-6.843E+00	-6.212E+00	-5.652E+00
	Max (m)	8.375E+00	7.661E+00	6.718E+00	5.907E+00	5.161E+00
Elcentro	Min (m)	-4.378E-01	-3.026E-01	-2.231E-01	-1.708E+01	-1.359E-01
	Max (m)	0	0	0	0	0
Santa Monica	Min (m)	-9.350E+00	-7.893E+00	-6.855E+00	-6.218E+00	-5.657E+00
	Max (m)	7.484E+00	6.413E+00	5.833E+00	5.529E+00	5.249E+00
Petrolia	Min (m)	-9.161E+00	-7.774E+00	-6.741E+00	-6.144E+00	-5.586E+00
	Max (m)	5.185E+00	5.048E+00	5.043E+00	4.934E+00	4.835E+00
Pomona	Min (m)	-6.235E+00	-5.208E+00	-4.521E+00	-4.090E+00	-3.718E+00
	Max (m)	4.649E+00	3.826E+00	3.522E+00	3.296E+00	3.106E+00

The reinforcement in the bridge's capping beam was varied when the columns were reinforced with 100% Steel for the Newhall earthquake being the scenario with the highest intensity and the results are presented in Figure 11. The results revealed that the introduction of SMA in the capping beam reduced its maximum displacement by 35% when reinforced with 75% steel and 25% SMA, 52% when reinforced with 50% steel and 50% SMA, 62% when reinforced with 25% steel and 75% SMA and 17% when reinforced with 100% SMA. The minimum allowable displacement for a beam is $^{Span}/_{500}$ [12]. In the case of the capping beam for the Matsurube Bridge, the minimum allowable beam deflection is 20mm. This is satisfied by all the combinations, but the minimum displacement at the midspan of the beam with a value of 6.03mm was obtained when the capping is reinforced with 25% steel and 75% SMA.

The results from the column displacement and displacement of the capping beam shows that the introduction of SMA in the bridge contributed to the significant difference in the results obtained for displacements in the columns and the capping beam.

5. Conclusion

This research has investigated the appropriate percentage of shape memory alloy combined with steel reinforcement that will give the least displacement for the percentage variation of SMA and steel reinforcements investigated in the column and the capping beam of a three-span composite bridge subjected to seismic dynamic load. Shape memory alloy shows a high resistance to seismic loads when combined with steel reinforcement and it is therefore recommended for inclusion in reinforced concrete bridges to serve as means of reducing the effect of earthquakes on structures in earthquake prone areas.

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