Design of bridges to satisfy ductility requirements

Ergin Atimtay[1]

ABSTRACT

Design codes permit plastic hinging of bridge piers under the design earthquake, provided that collapse does not occur; that is enough ductility must be available. However, ductility is not quantified and "How much ductility is enough ductility?" remains as a basic question for the designer to answer.

The ductility of the cross-section of a reinforced concrete member is obtained by developing the thrust-moment-curvature relationship. Then, a plastic analysis is performed to determine the sway of the supurstructure at the initiation of yielding. Thus, curvature and sway are related to each other. Now, the designer can decide on the desired sway ductility and calculate the corresponding curvature ductility. Then, the dynamic analysis of the bridge is performed in two steps, first without and then with abutment-superstructure interaction. The ductility demand thus obtained is compared with the available ductility. Obviously, available ductility must be greater than the demand. If not, design parameters must be changed to satisfy this constraint.

INTRODUCTION

Satisfying ductility requirements has been an essential goal of design against earthquakes. According to AASHTO (1983), bridges must resist small to moderate intensity ground shaking in the elastic range. For design earthquakes, the design philosophy accepts plastic hinging in the piers, provided that collapse does not occur. AASHTO further requires that such plastic hinging in the piers must be repairable. Damage to the foundations and joints is not acceptable.

However, design codes do not really define how the essantial ductility requirements can be met. The amount of confinement, splice requirements, etc. are carefully stated, but the following questions still ask for answers in the designer's mind: How much ductility is provided by a certain amount of confinement in a cross-section with a certain geometry? How can the elusive concept of ductility be quantified? Given a structural system, how much ductility is to be provided?

(I) Prof.Dr.Building Science, Middle East Technical University, Ankara, TÜRKİYE.

CROSS-SECTIONAL DUCTILITY

The cross-sectional ductility of a reinforced concrete member is commonly expressed by the thrust-moment-curvature (N-M-K) relationship. N-M-K relationships are obtained by solving a set of equations which are dictated by equilibrium, the proper stress-strain relationship, and compatibility conditions. The degree of confinement of the cross-section can be accounted for by modifications in the stress-strain relationship of concrete.

A computer program to give the N-M-K relationship has been developed (Atimtay, Cengizkan, 1975), which is another addition to many available. It's validity has been checked against test data and found to be very acceptable.

Being underreinforced, reinforced concrete beams exhibit a definite yield point. However, with the application of the axial load, the N-M-K- relationship begins to get non-linear and the definite yield moment gradually disappears. For bridges in earthquake zones, the level of axial load is kept small, usually under 0.2. At such axial load levels, the yield point of the member can still be defined easily on the N-M-K relationship. A typical N-M-K relationship for a bridge pier subject to low axial load levels is shown in Fig.1.

DUCTILITY DEMAND

It is common to design bridges with a gap between the superstructure and the abutment. This enables the superstructure to displace longitudinally under temperature effects and small to moderate intensity ground shaking.Additionally the abutment can be designed as free-standing where only active soil pressure behing the abutment can be considered under service conditions.

If designed in conformance with the AASHTO requirements, the bridge piers will hinge under the design earthquake, thus magnifying the superstructure sway. As a result, the gap between the superstructure and the abutment will close. The superstructure will bang against the abutment wall and the passive soil resistance behind the wall will be mobilized. Consequently, this mobilized passive soil pressure will restrain the superstructure sway. For the above described behavior to take place, the bridge pier will have to hinge at the pier-footing connection putting a substantial demand on ductility. To be able to meet this demand in the structural design context, ductility must be quantified. This requirement leads to the following question: Given a certain longitudinal sway, how much ductility must be provide so that the AASHTO requirements of earthquake damage and repairability can be satisfied?

The elevation of a typical bridge is shown in Fig.2. Seat type abutments have been employed to allow free movement of the superstructure under temperature effects and small to moderate intensity ground shaking. The superstructure may be continuous or simply supported over the piers. The piers act as supports for the superstructure and no moments are transferred between the superstructure and the piers. This bridge type is very common and popular, where the contractor slip-forms the piers and the superstructure is prefabricated. Once the piers are completed, the superstructure is dropped in place (or pushed-out) and seismic keys are cast-in-situ to prevent lateral movement of the superstructure.

The piers may be multi-column bents or wall-type piers (blade piers).The contractors prefer wall-type piers because they can be slip-formed much more easily. Preferable they may be from a contractor's point of view, pier walls present special problems for the designer.

Under the design earthquake, the only location in the pier wall where hinging can occur is the pier-footing connection. Obviously, with hinges at the pier-footing connections, the structure will be transformed into a mechanism. Consequently, the superstructure will sway until it bangs against the abutment back wall.

DESIGN PROCEDURE

The above described behavior will necessitate a two-phase analysis procedure. Firstly, the bridge must be considered as a structure which can sway freely without any interaction with the abutment. At this stage, the amount of sway and the gap between the superstructure and the abutments must be carefully monitored. Under moderate intensity ground shaking, the piers must not reach yielding. The author prefers to use a ratio of "service load moment" to yield moment, $M_s/M_y=0.7$. In other words, the maximum moment at the pier-footing connection, which is developed when the bridge makes longitudinal sways under a moderata intensity earthquake, should not exceed 70% of the yield moment of the pier.

In the second phase, the bridge must be considered as a system which interacts with the abutment back wall. The passive soil pressure behind the abutment wall may be considered as elastic springs which stabilize the superstructure sway. Here, the designer is faced with a tough question to answer: What is the "spring constant equivalent" of the select fill material behind the back wall which is in passive pressure mode?

Caltrans (1987) makes suggestions about the stiffness coefficients for average abutment backfill conditions:

k_s = Soil stiffness per linear length of wall
= 36 kN/m (200 k/in) based on material with
V_s = 0.3 m/sec.
(effective height of wall $\mp$ 2500 mm)

It is clear to the engineer that the soil behind the back wall cannot sustain sresses after certain limits are exceeded. Here again, Caltrans recommendation will be followed, that maximum effective soil stress of 375 kN/m^2, should not be exceeded. The preliminary abutment stiffness should be adjusted after dynamic analysis, if the resulting forces on soil or displacement of the abutment are found to be excessive.

The structural model after the superstructure makes contact with the abutment back wall is shown in Fig.3. When the structure sways under the design earthquake, it hits the left abutment back wall mobilizing the full passive pressure. When it reverses the sway and moves away from the abutment, no soil pressure acts on the structure. Then, it hits the right abutment back wall and the full passive pressure is again mobilized. To account for this behavior, one-half of the total soil resistance is considered at each abutment. This is necessary to compute the correct dynamic properties of the bridge system. However, attention should be paid to the point that the resulting forces from the analysis should be doubled when designing the abutments.

QUANTIFYING DUCTILITY

Consider a pier as shown in Fig.4. with the corresponding moment and curvature distributions (Park and Paulay, 1975). The displacement of the pier at its top can now be calculated.

$$\Delta_u = \left(\frac{Ky}{2} \cdot \frac{2L^3}{3}\right) \cdot (Ku - Ky)\ Lp\ (L - 0.5\ L_p) \qquad (1)$$

$$\Delta_y = \frac{Ky}{2} \cdot \frac{2L^3}{3} \qquad (2)$$

Ky = curvature corresponding to yielding
Ku = curvature at the end of the post-yielding range
Δu = displacement at the end of the post-yielding range
Δy = displacement corresponding to yielding
L = length of the pier
Lp = equivalent length of the plastic hinge

The displacement ductility ratio μ and curvature ductility ratio Ku/Ky can be calculated as follows :

$$\mu = \frac{\Delta_u}{\Delta_y} = 1 + \frac{(Ku - Ky)}{Ky} \cdot \frac{Lp\ (L - 0.5\ Lp)}{L^2/3} \qquad (3)$$

$$\frac{Ku}{Ky} = \frac{L^2\ (\mu - 1)}{3Lp\ (L - 0.5\ Lp)} \qquad (4)$$

Earthquake resistant structures should posses a displacement ductility ratio of at least 4. So, assuming $\mu = 4$, Table 1 can be prepared to give the curvature ductility demands. It should be noticed that Lp/L is a variable. In other words, the length of the plastic hinging influences the ductility demand greatly.

Since the equivalent length of the plastic hinge Lp is typically in the range of 0.5 - 1.0 times the member depth, the length of the pier closely controls the curvature ductility demand. For very high piers, Lp/L will be small, thus creating the greatest demand on ductility. This demand should be met by the available ductility as given by the N-M-K relationship.

CONCLUSION

A design method has been developed which considers the interaction of the superstructure with the abutments. The total longitudinal sway of the structure puts a demand on ductility of the pier at the pier-footing connection. This ductility demand is quantified. The design method also shows how this quantfied ductility demand can be met by making use of the thrust-moment-curvature relationship of the pier. Cross-sectional properties can be changed as necessary to provide the required ductility.

REFERENCES

1. "Guide Specifications for Seismic Design of Highway Bridges (1983)", American Association of State Highway and Transportation Officials, 1983, 11pp.

2. "Bridge Memo to Designers Manual, 1987", State of California, Department of Transportation, 1987, 10-15 pp.

3. Atimtay, Ergin and Cengizkan, Kemal, "A Numerical Approach to Creep of Reinforced Concrete", METU Journal of Pure and Applied Sciences, Vol.8, No.3, 1975, 365 pp.

4. Park, Robert and Paulay, Thomas, Reinforced Concrete Structures, John Wiley and Sons, New York, 1975, 569 pp.

APPENDIX-NUMERICAL EXAMPLE

The typical underpass as shown in Fig.5 will be designed.
Width of superstructure = 11500 mm
Width of abutment = 11000 mm
Weight of superstructure = 237 kN/m
Select pier cross-section as shown in Fig.5.
Normal force on pier = 8053 kN
Develop N-M-K relationship of the pier for materials C20 and S420, as shown in Fig.6. This will provide the cracked EI, the moment capacity of the pier and the corresponding Ky.
Perform intermediate level eartquake analysis by the Response Spectrum Method.
Use effective EI (AASHTO Eq. 4-1) in dynamic analysis.
Superstructure sway = 63 mm
Max. pier moment = 17000 kN-m
M/Mu = 0,67 (about 70%, say OK)
Change pier geometry if unacceptable.

$$\Delta_y = \frac{Ky}{2} \cdot \frac{2L^3}{3} = \frac{5,02 \times 10^{-3}}{2} \cdot \frac{2\ (7)^2}{3} = 0,0849 \text{ m} = 84,9 \text{ mm}$$

Choose $\mu = 4$
Total Δu = 84,9 x 4 = 339,6 mm
Check if this total sway can be sustained by the pier

Maximum displacement of the pier must be 75 mm (CALTRANS).
Perform design eartquake analysis considering the passive pressure of the soil behind the abutment back wall as equivalent elastic springs (Fig.3).
Soil stiffness = 1264000 kN/m (CALTRANS)
Effective soil strength = 9525 kN (CALTRANS).
Abutment movement = 75.8 mm $\sim$ 75 mm OK.
Total EQ force = 7582 kN
Total EQ force on pier = 3714 kN (AASHTO 4.8.2)
Do not use overstrength factor to calculate pier moment capacity when checking if soil strength is exceeded or not.
Total force on abutment wall = 7582 - 3714 = 3868 kN $<$ 9525 kN
The constraints related with the abutment movement and soil strength are satisfied.
Check is enough ductility is provided in the pier to sustain a total longitudinal displacement of 339.6 mm.
Take length of plastic hinge $L_p = 0.65(b) = 0.65 \times 1.0 = 0.65$ m.
$L_p/L = 0.65/(7-1) = 0.108$ (Considering hinge lengths)
$K_u/K_y = 11.0$ (by interpolation, Table 1)
Calculate curvature ductility from N-M-K of pier (Fig.8).
$K_u/K_y = 10.67 < 11.0$ but close. Say OK.

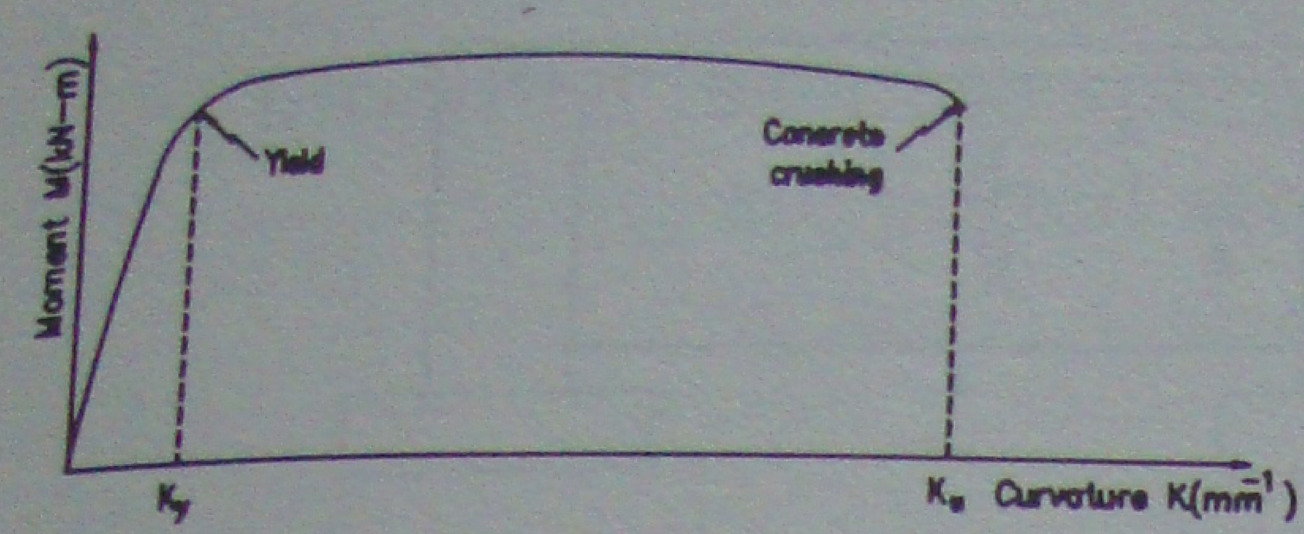


Figure 1. Typical N-M-K relationship for low axial load levels.

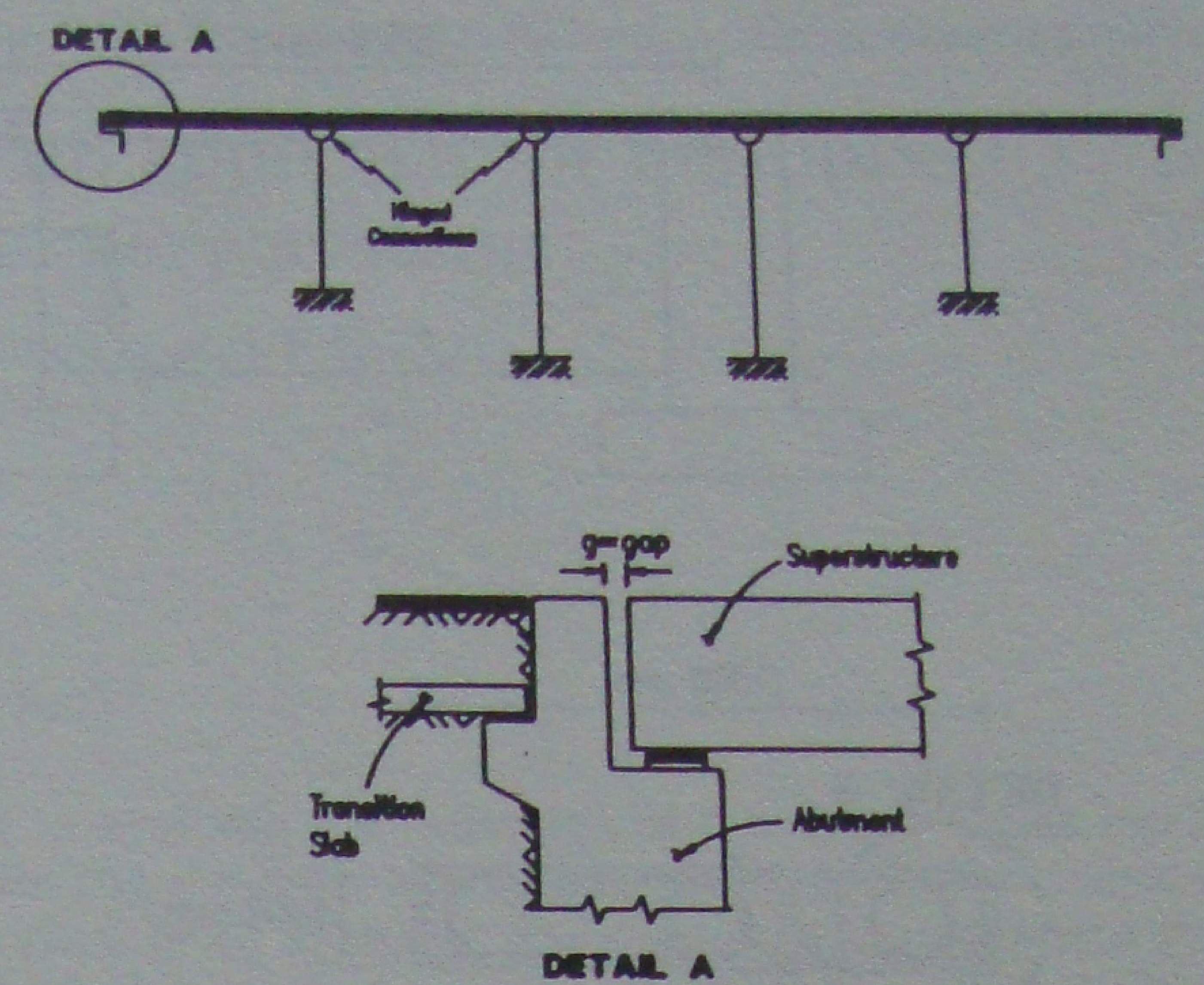


Figure 2. The elevation of a typical bridge.

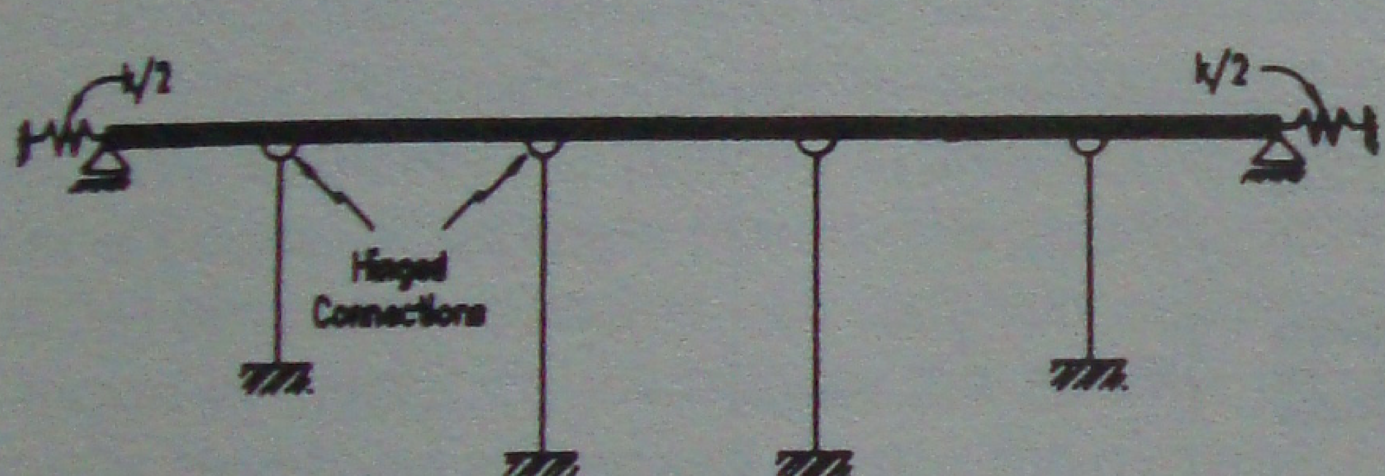
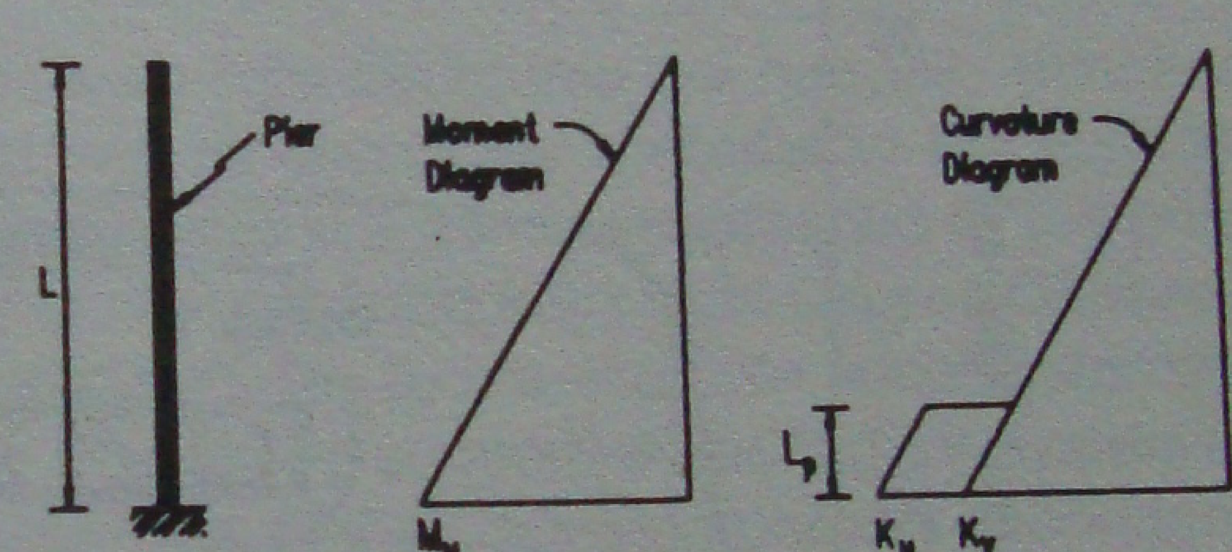


Figure 3. The structural model of bridge-abutment interaction.



Fifure 4. The pier at ultimate condition (Ref.3)

Table 1. Ductility demands for $\mu = 4$

L_p / L	0.05	0.1	0.15	0.20	0.25	0.30	0.35
K_u / K_y	21.5	11.5	8.2	6.6	5.6	4.9	4.5

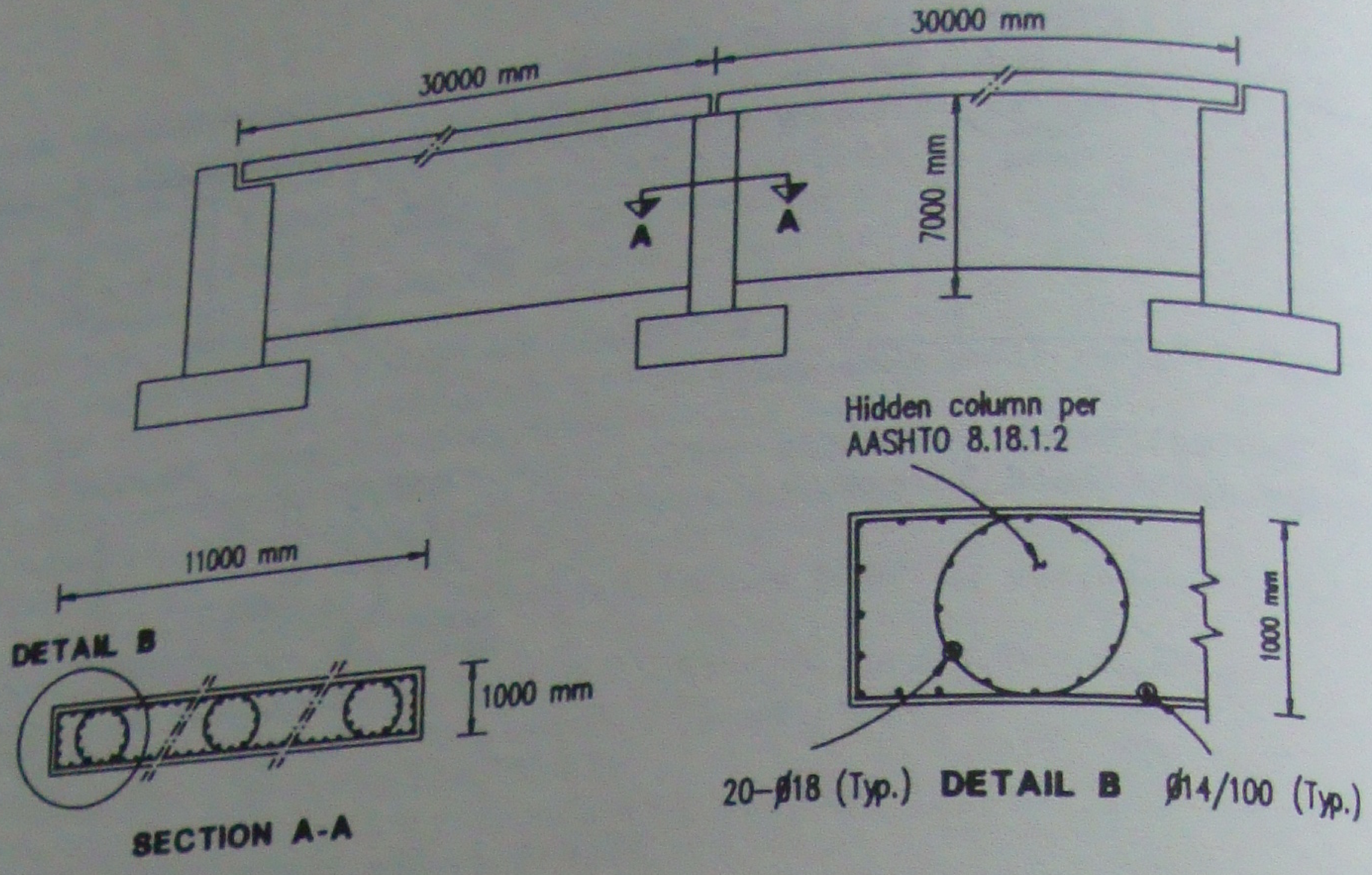


Figure 5. Typical underpass of the design example.

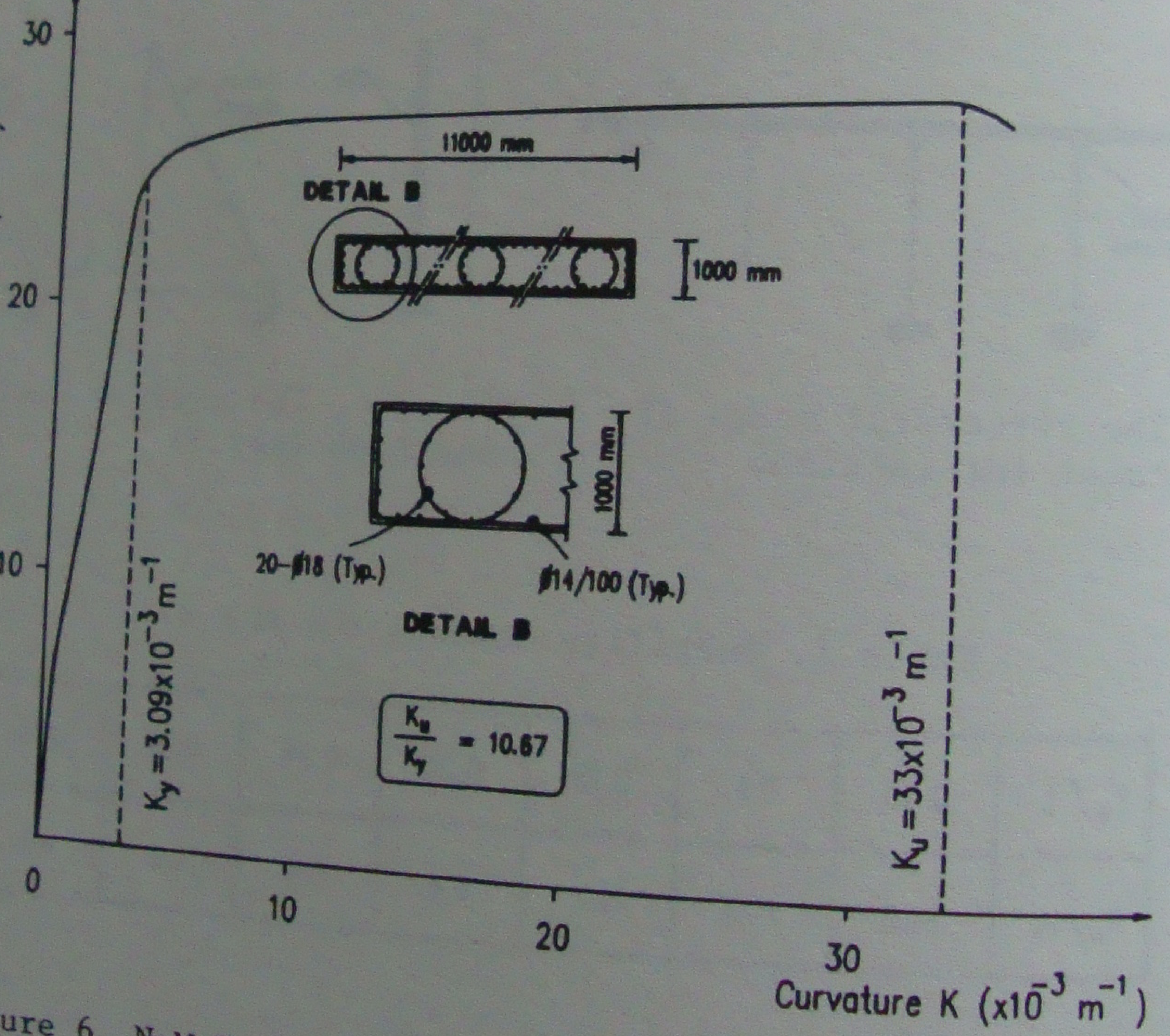


Figure 6. N-M-K relationship of pier used in the example.