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Response of reinforced soil slopes to earthquake loadings

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ABSTRACT

Principal features of a direct nonlinear method for analyzing reinforced soil structures under earthquake loading is presented. The response of a soil slope reinforced with horizontal layers of polymeric reinforcement and subjected to a typical earthquake loading is presented. The seismic response of the unreinforced soil slope is also included to examine the influence of the reinforcement.

INTRODUCTION

The use of advanced polymeric materials as tensile soil reinforcement in the construction of embankments and slopes with side slopes steeper than the angle of internal friction of the soil fill is becoming more common. Conventional limit equilibrium–based methods are commonly used for analysis and design of these slopes (Bonaparte et al. 1986). These methods are primarily stress–based and do not consider deformations explicitly. Moreover, little attention has been given to the design of reinforced soil slopes subjected to seismic loading.

In recent years, however, the use of finite element analysis has been introduced into the study of the dynamic response of reinforced soil systems to seismic load. The study reported here presents a direct nonlinear method for analyzing reinforced soil structures under dynamic load conditions. The essential features of the method are implemented in the finite element program TARA–3 (Finn et al. 1986). This program has been used successfully to analyse the seismic response of a centrifuged model of a cantilever retaining wall (Finn et al. 1989) on the Cambridge geotechnical centrifuge. The program simulated satisfactorily not only the accelerations, dynamic displacements and dynamic moments but also the residual displacements and moments remaining after the earthquake.

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TARA–3 has also been used to investigate key aspects of the seismic response of a reinforced soil retaining wall (Yogendrakumar et al. 1991). In this paper, the same method of analysis reported in these earlier studies is used to examine the influence of polymeric reinforcement on the seismic response of a soil slope.

METHOD OF ANALYSIS

This section presents the principal features of the direct nonlinear method that is implemented in the finite element program TARA–3 for total stress analysis of soil structures subject to dynamic loading. In this method an incremental approach has been adopted to model nonlinear behaviour of soil using tangent shear and tangent bulk moduli, G_t and B_t respectively. The incremental displacements during the base excitation are obtained by solving the incremental dynamic equilibrium equations given in Eq. 1 by a direct numerical integration method.

$$[M]\{\Delta\ddot{x}\} + [C]\{\Delta\dot{x}\} + [K]\{\Delta x\} = -[M]\{I\}\Delta\ddot{u}_g \tag{1}$$

Here [M] is the mass matrix; [C] is the damping matrix; [K] is the stiffness matrix; {I} is the unit vector; $\{\Delta\ddot{x}\}$, $\{\Delta\dot{x}\}$ and $\{\Delta x\}$ are incremental acceleration, velocity and displacement vectors of the nodes relative to the base and; $\Delta\ddot{u}_g$ is the increment in base input acceleration.

The stiffness matrix [K] is a function of the current tangent moduli during loading, unloading and reloading. The use of shear and bulk moduli allows the elasticity matrix [D] to be expressed as

$$[D] = B_t\,[Q_1] + G_t\,[Q_2] \tag{2}$$

where $[Q_1]$ and $[Q_2]$ are constant matrices for the plane strain conditions usually considered in the analysis. This formulation reduces the computation time for updating [D] whenever G_t and B_t change in magnitude because of straining.

<u>Soil model</u>

The behaviour of soil in shear is assumed to be nonlinear and hysteretic and to exhibit Masing behaviour during unloading and reloading. The relationship between shear stress τ and shear strain γ for the initial loading phase under either drained or undrained loading conditions is assumed to be hyperbolic and given by

$$\tau = f(\gamma) = \frac{G_{max}\,\gamma}{\left[1 + (G_{max}/\tau_{max})|\gamma|\right]} \tag{3}$$

where G_{max} is the maximum shear modulus and τ_{max} is the maximum shear strength. The initial loading curve is shown in Fig. 1a. The equation for the unloading curve from a point (γ_r, τ_r) at which the loading reverses direction is given by

$$\frac{\tau - \tau_r}{2} = f\left(\frac{\gamma - \gamma_r}{2}\right) \tag{4}$$

or

$$\frac{\tau - \tau_r}{2} = \frac{G_{max}\ (\gamma - \gamma_r)/\ 2}{\left[1 + (G_{max}/\ 2\tau_{max})\left|\gamma - \gamma_r\right|\right]} \tag{5}$$

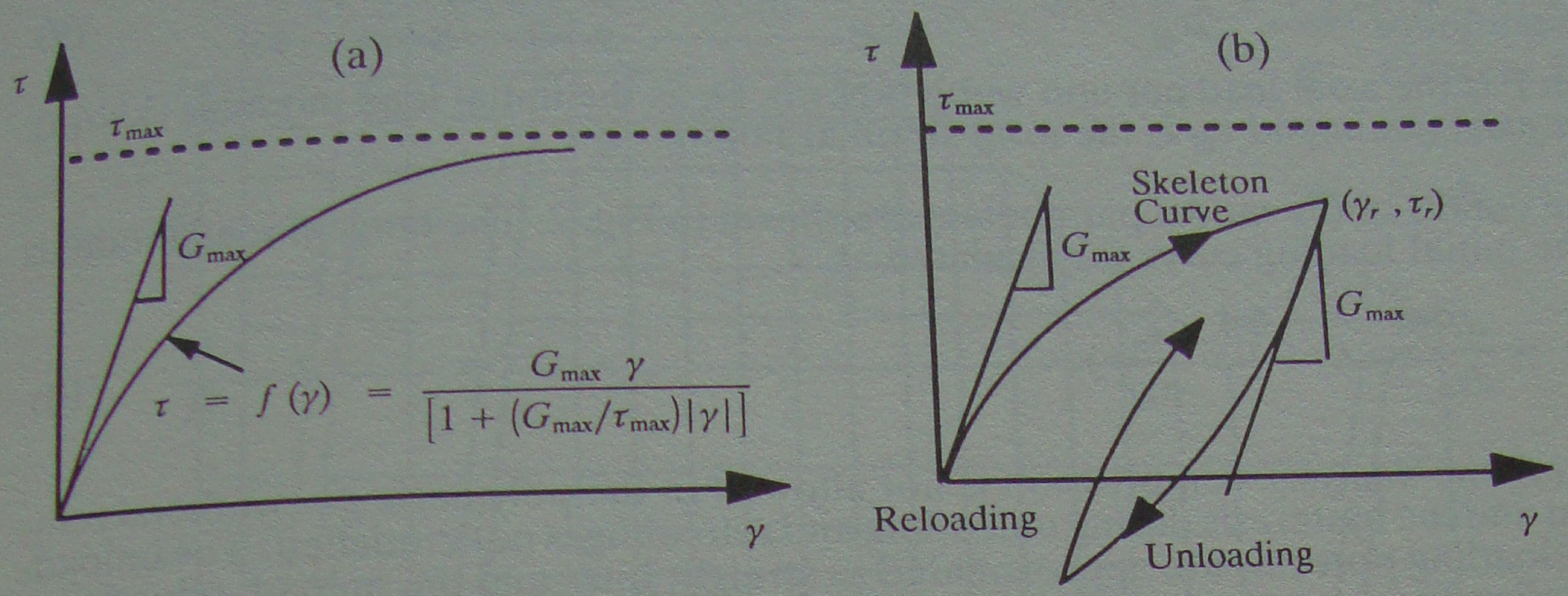

Figure 1. Nonlinear hysteretic loading paths

The shape of the unloading–reloading curve is shown in Fig. 1b. The tangent shear modulus, G_t, for a point on the skeleton curve is given by

$$G_t = \frac{G_{max}}{\left[1 + (G_{max}/\tau_{max})\left|\gamma\right|\right]^2} \tag{6}$$

and at a stress point on an unloading or reloading curve G_t is given by

$$G_t = \frac{G_{max}}{\left[1 + (G_{max}/\ 2\tau_{max})\left|\gamma - \gamma_r\right|\right]^2} \tag{7}$$

The response of the soil to uniform all–round pressure is assumed to be nonlinearly elastic and dependent on the mean normal stress. Hysteretic behaviour, if any, is neglected in this mode. The tangent bulk modulus, B_t, is expressed in the form

$$B_t = K_b\ P_a\ (\frac{\sigma_m}{P_a})^n \tag{8}$$

in which K_b is the bulk modulus constant, P_a is the atmospheric pressure in units consistent with mean normal effective stress σ_m and n is the bulk modulus exponent.

<u>Reinforcement model</u>

The reinforcement is modelled using one–dimensional beam elements with axial stiffness only. Slip elements of the type developed by Goodman et al. (1968) may be used to allow for the relative movements between the soil and reinforcement during earthquake excitations. Relatively inextensible type of reinforcement is assumed to be an elastic perfectly plastic material with the yield stress given by the elastic limit. The behaviour of relatively extensible reinforcement such as polymeric materials is assumed to be nonlinear. The relationship between axial load and axial strain for the initial loading is assumed to be of the nonlinear quadratic form (Chalaturnyk et al. 1988) given by

$$F = D_i \epsilon_a \left(1 - \frac{\epsilon_a}{2\epsilon_{af}} \right) \tag{9}$$

in which F is the axial load per unit width (kN/m), D_i is the initial load modulus (kN/m), ϵ_a is the axial strain and ϵ_{af} is the axial strain at failure.

The details of the model parameters are shown in Fig. 2. The tangent load modulus D_t on the initial loading curve is calculated as

$$D_t = \frac{dF}{d\epsilon_a} = D_i \left(1 - \frac{\epsilon_a}{\epsilon_{af}} \right) \tag{10}$$

During the analysis, compression is not allowed in the polymeric geosynthetic reinforcement and the hysteresis during unloading and reloading is not modelled. The unloading and reloading portions are approximated as straight lines and the unload–reload modulus is defined as

$$D_{ur} = K \ D_i \tag{11}$$

in which D_{ur} is the unload–reload modulus and K is a constant.

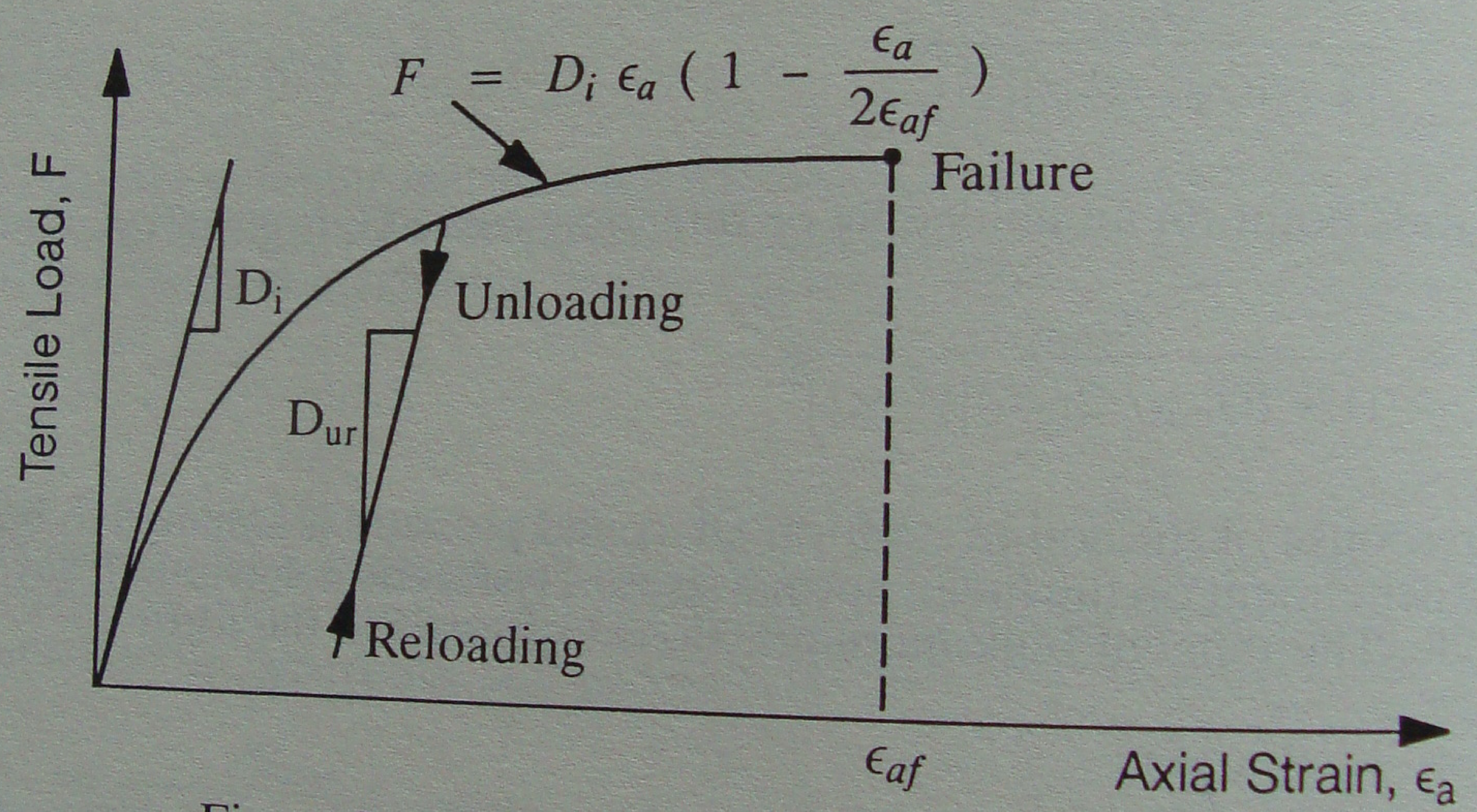

Figure 2. Nonlinear load–strain relationship

FINITE ELEMENT ANALYSIS

The dynamic responses of a reinforced and unreinforced soil slope resting on a rigid foundation were computed using the TARA-3 program. The slopes were assumed to be 12.0m high with a side slope of 1:1. It is lightly reinforced with polymeric reinforcements 12.0m in length. The reinforcement layers are placed horizontally with vertical spacing of

2.0m. The finite element representation of the reinforced soil slope shown in Fig. 3 consisted of 87 soil elements and 30 one–dimensional beam elements in the slope and 69 elements in the foundation. Slip elements which allow for relative movement to occur between the soil and the reinforcement have not been used in this analysis.

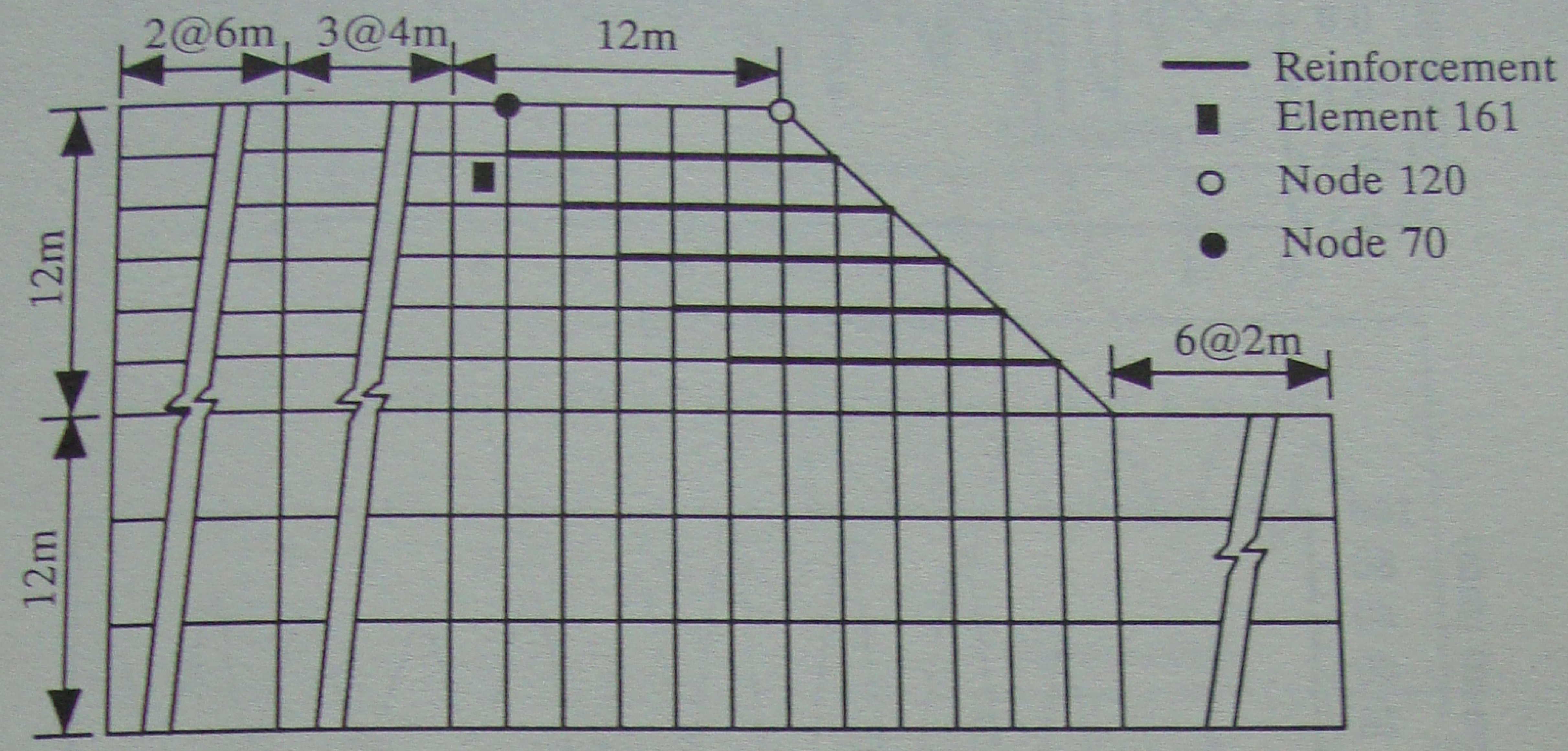

Figure 3. Finite element representation of the reinforced soil slope

The foundation soil was assumed to be very stiff and the shear modulus, Poisson's ratio and unit weight were taken as 3500 MPa, 0.49 and 20.0 kN/m^3 respectively. The following properties were selected for the soil in the slope; K_b = 2950.0, n = 0.5, Poisson's ratio = 0.40, cohesion = 35 kPa and angle of internal friction = 17°, unit weight = 20 kN/m^3. For the polymeric reinforcement, D_i, ϵ_{af} and K were taken as 778.0, 0.18 and 2.0 respectively. The response of the slope to the first 9.60 seconds of the N–S component of the 1940 El Centro earthquake scaled to 0.2g was computed using the program TARA–3. The input motion is shown in Fig. 4. The base was assumed to be rigid and the nodes on the left and right vertical boundary were supported on horizontal rollers for the dynamic analysis. A static analysis was first conducted to establish the stress–strain field prior to the earthquake excitation. The program simulated the incremental construction process of the slope.

NUMERICAL RESULTS

Fig. 5 shows the dynamic horizontal displacement time history of node 70 for the unreinforced and reinforced slopes. Node 70 is located at the top surface 10m from the crest of the slope. Because the nonlinear behaviour of both the soil and the reinforcement is modelled, the analyses show that there is residual dynamic displacement present after the earthquake in both the reinforced and unreinforced slopes. The comparison in Fig. 5 clearly shows the effect of the reinforcement in reducing slope deformations. The peak values and the size of oscillations of the displacements are reduced in the case of the reinforced slope. For this node, the maximum and the residual displacements are reduced to about 11% and 20% respectively of the values predicted for the unreinforced slope.

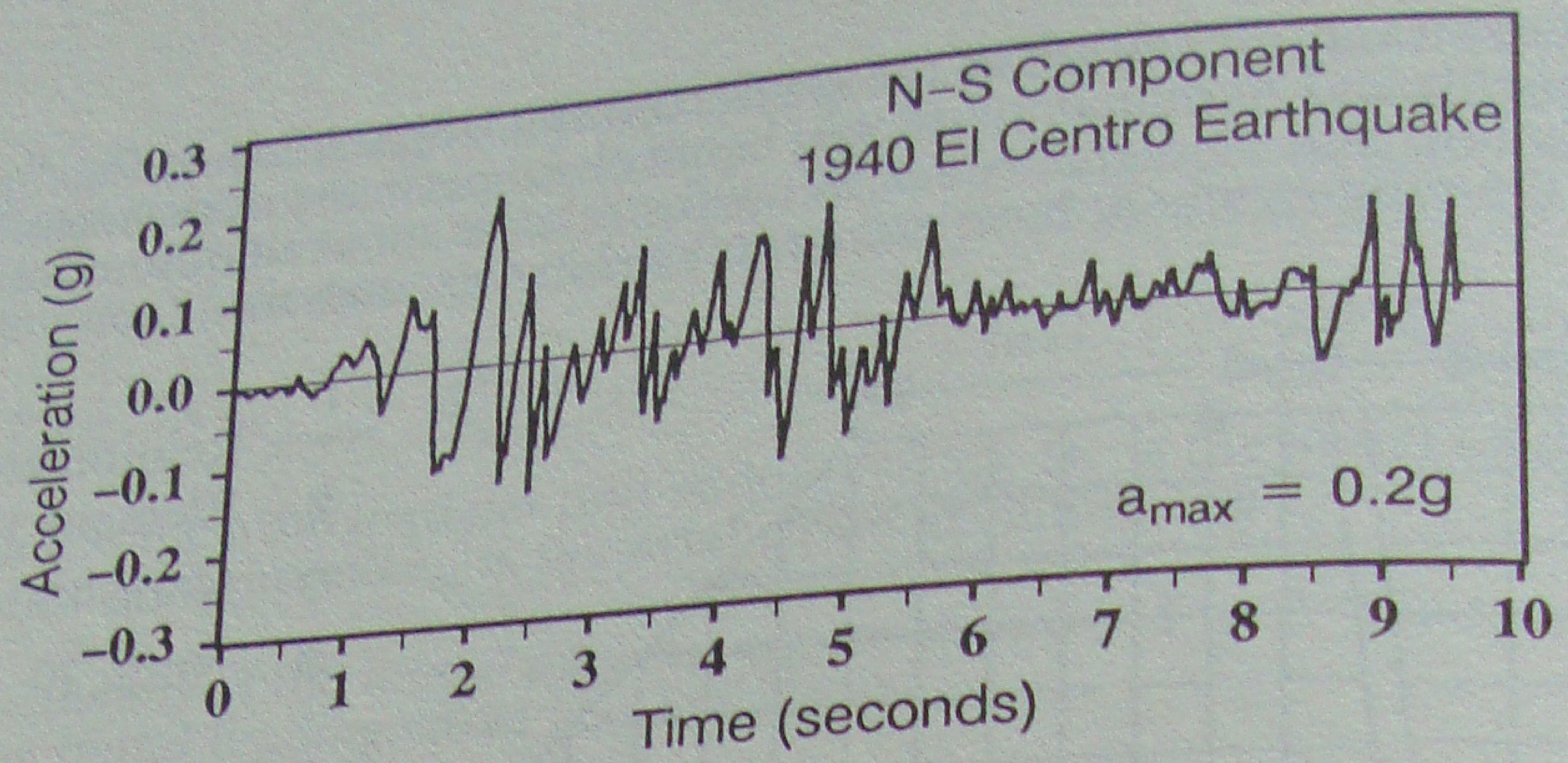

Figure 4. Base input motion for TARA–3 analysis

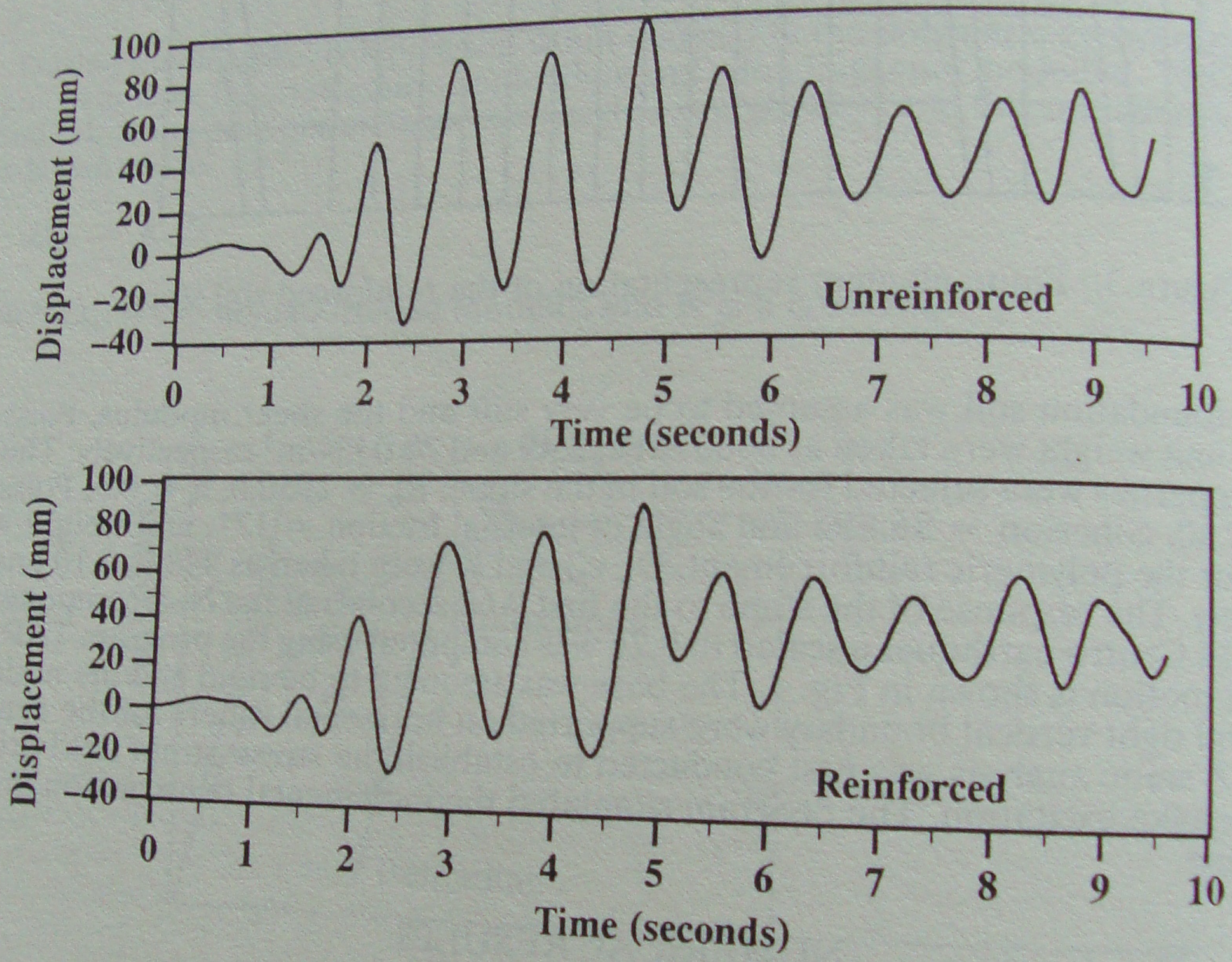

Figure 5. Displacement time history of node 70

The dynamic horizontal displacement time histories of node 120, the node at the top edge of the side slope, are shown in Fig. 6. Again as expected, analysis with the reinforced slope produces smaller displacements than the analysis of the unreinforced slope. The maximum displacement in the case of reinforced slope is 10% less than value with the unreinforced slope. A reduction of about 30% is achieved in the residual displacement. Similar reductions in maximum and residual deformations are observed at other locations within the soil slope indicating the beneficial effect of the reinforcement.

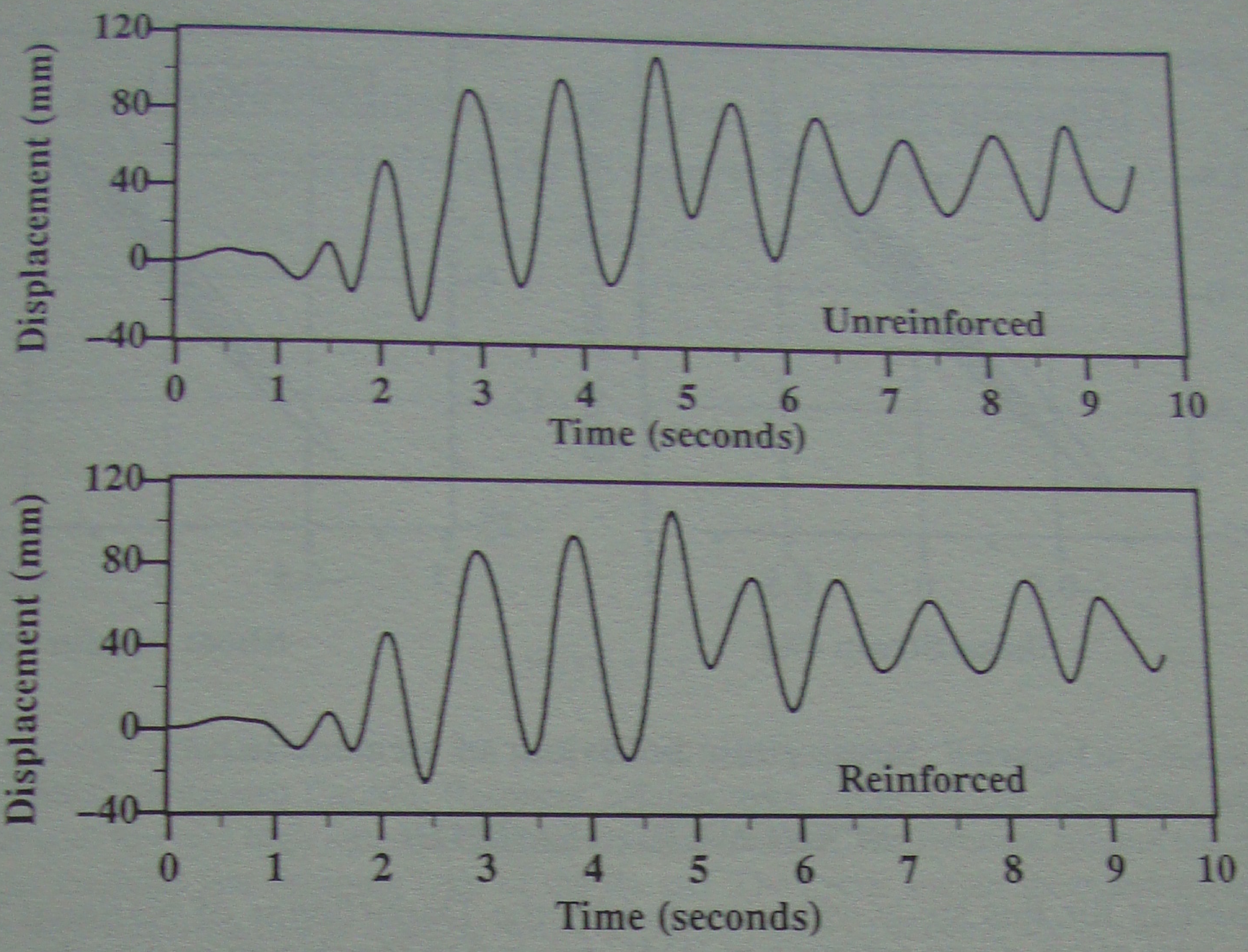

Figure 6. Displacement time history of node 120

The improved seismic performance of the reinforced slope is further illustrated in Fig. 7. This shows the shear stress–strain response of soil element 161 for the case of unreinforced and reinforced soil slope. Both show nonlinear response with stronger response occurring in the case of the unreinforced slope. The magnitude of the strain and the size of hysteresis loops are larger in the case of unreinforced slope than in the case of reinforced slope.

CONCLUSIONS

The TARA–3 analysis is capable of providing the key information needed to assess the seismic performance of a soil slope in terms of deformations. Because the nonlinear hysteretic behaviour of soil and the nonlinear behaviour of the polymeric reinforcement are modelled, the residual deformations after the earthquake can be calculated directly and the consequences of seismic shaking clearly seen.

An important concern is the contribution of the polymeric reinforcement in improving the seismic behaviour of the soil slope. This is readily determined by TARA–3 analysis as the program has the capability to model the nonlinear load–strain behaviour of the polymeric reinforcement. For the example considered it has been shown that the polymeric reinforcement has improved the seismic behaviour of the soil slope.

The results so far suggest that the program TARA–3 may be a very useful tool in assessing the seismic performance of reinforced soil slopes. Studies are planned to determine the influence of the different model parameters describing the nonlinear load–strain behaviour of the reinforcement.

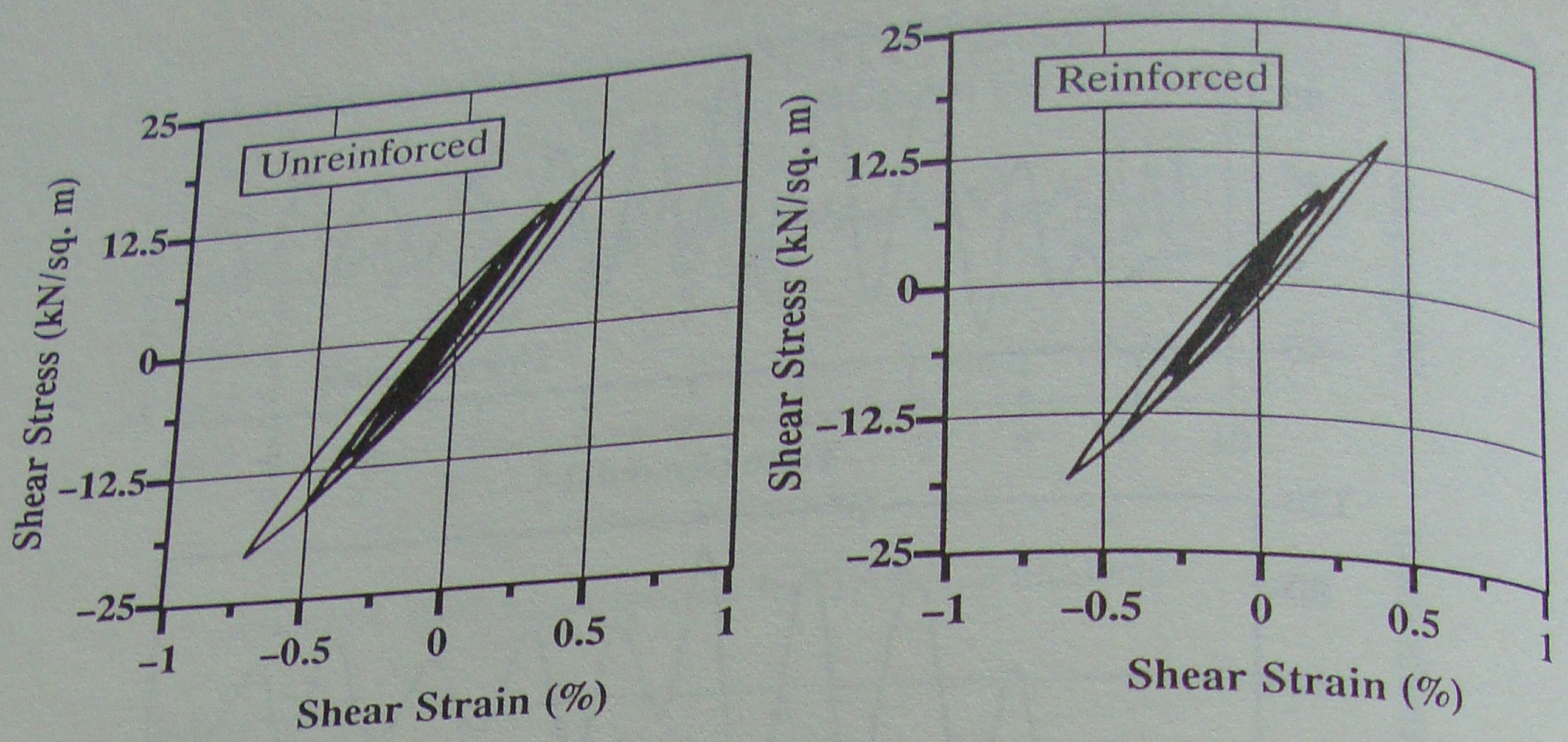

Figure 7. Stress–strain behaviour of element 161.

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