Permanent deformation prediction modelling of asphalt concrete layers based on Accelerated Pavement Testing

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ABSTRACT: One of the main distresses that Mechanistic – Empirical (M-E) design methods attempt to control is permanent deformation (rutting). The objective of this paper is to evaluate an M-E permanent deformation model for an asphalt layers using test results from accelerated pavement testing. Two series of Heavy Vehicle Simulator (HVS) test structures were used. The permanent deformation, wheel loading, pavement temperature, and other material properties were continuously controlled during the HVS testing. The performance measurements were made at 10, 20 and 30°C pavement temperatures. Asphalt concrete layers were considered as linear elastic and their stiffness was modified according to the loading temperature where as stress dependent behaviour of unbound materials was considered when computing input parameters (stress and strains) for the M-E permanent deformation model. The traffic wandering was also accounted in modelling the traffic assuming normal wander distribution and a time hardening approach was applied to accumulate the permanent deformation contributions from different stress levels. The measured and predicted permanent deformations are in general in good agreements for 10 and 20°C temperatures however for a temperature of 30°C, the M-E model over-predicted the permanent deformations behaviour thus a better characterization of material properties and more validation is needed.

KEY WORDS: Asphalt concrete, rutting, Heavy Vehicle Simulator, traffic wandering, Mechanistic - Empirical, response model.

1 INTRODUCTION

Most of the paved roads in Sweden are flexible pavements, consisting of unbound granular base and asphalt concrete layers. Flexible pavements are particularly subject to permanent deformation (rutting) and fatigue failures in the asphalt layers. Apart from traffic loading, pavements are subjected to varying environmental conditions which eventually lead to an increase in temperature in the asphalt layers and moisture content in the base and subgrade soils, resulting in a decrease in stiffness's of the different pavement layers. Repeated vehicle loading under these weakened conditions can lead to rutting and fatigue cracking in the asphalt layers (FHWA, 2003).

Prediction of rutting along with other distress modes became the concern of modern flexible pavement design procedures as opposed to the empirical approaches which have been utilized for many years.

Mechanistic Empirical (M-E) procedures provide a more realistic characterization of pavement structures and the related variables. In the M-E schemes the pavement structure is analyzed for the prevailing traffic loads and environmental conditions and the procedure attempts to limit the stresses and deformations or damages in the pavement structure to a desirable tolerances. M-E approaches thus in general have two phases; computing stresses using response model and empirically relating the computed responses with the expected pavement life or pavement performance. The repeated applications of traffic loads on the pavement structures makes the empirical approach inevitable for relating responses to the deformations or damage. However the M-E approach has far better merits than the pure empirical design methods in that it is possible to predict quantitatively the different types of distresses or damages to the best possible levels of tolerances.

The need for introducing new pavement construction materials also requires performance assessment of the materials in question. This makes the use of empirical methods doubtful as these methods are generally established for commonly used pavement materials. With the M-E design methods it is possible to introduce new construction materials through proper material characterization. Performance modelling is also important part in assessing the life cycle cost a new or rehabilitated pavement structure (Santos and Ferreira, 2013).

The objective of this paper is to evaluate the permanent deformation model for asphalt layers used in the US Mechanistic Empirical Pavement Design Guide (MEPDG) by using results of Heavy Vehicle Simulator (HVS) tests performed on two test structures with conventional and modified binder base layers respectively. The permanent deformation model was implemented in an M-E performance prediction computer program (Ahmed and Erlingsson, 2012 and 2013).

2 PERMANENT DEFORMATION MODELING

As pavements are composed of several layers of different materials, it is hardly possible to find a single analytical model that can estimate the permanent deformation of the whole pavement structure due to prevailing traffic and climate conditions. Thus in this paper a layer strain model (Barksdale, 1972), shown in Equation 1, has been utilized. In this approach, the permanent deformation in each constituent pavement layer is computed from a corresponding permanent deformation model using a set of respective material properties. Once the permanent strain within each pavement layer is calculated, the total rut depth on the pavement surface can then be obtained by summing up the contributions of permanent deformation from each layer. This approach can also be extended for cases with thick pavement layers by further dividing the layers into sub-layers. Mathematically this procedure is given as:

$$R_d = \int_0^H \varepsilon_p(z)dz = \sum_{i=1}^{n} \varepsilon_{p,i}\Delta z_i \tag{1}$$

where $\varepsilon_{p,i}$ and Δz_i denote permanent strain and the thickness of the i^{th} sub-layer respectively, n is the total number of sub-layers, and R_d is the rut depth, H is the depth from the pavement surface to rigid bottom or rock layer. ε_p is estimated using M-E permanent strain models. In this paper, the permanent deformation model for asphalt layers which is implemented in the MEPDG was used.

The MEPDG model which is a function of the vertical strain in the asphalt layer, the pavement temperature and the number of load repetition and it is given by (ARA, 2004):

$$\hat{\varepsilon}_p = a_1 T^{a_2} N^{a_3} \varepsilon_r \tag{2}$$

where $\hat{\varepsilon}_p$ is the accumulated permanent strain, a_1, a_2 and a_3 are material constants, T is the pavement temperature, N is the number of load repetition, and ε_r denotes resilient vertical strain calculated using a response model.

The responses (stresses and strains) were calculated using a program called ERAPAVE (Erlingsson and Ahmed, 2013) which was developed as a part of an M-E performance prediction program (Ahmed and Erlingsson, 2012 and 2013; Erlingsson, 2012). The response program (ERAPAVE) considers nonlinearity (stress sensitivity) of unbound granular materials. The asphalt layers are considered as linear elastic materials.

3 ACCELERATIVE PAVEMENT TEST (APT) STRUCTURE

In this study, test pavement structures tested using a Heavy Vehicle Simulator (HVS) are considered. HVS is a mobile equipment and can be used to study the behaviour of pavement structure under wheel loading and environmental conditions which are close to actual field conditions. The HVS facility owned by the Swedish Road and Transport Research Institute, Sweden was employed for the study. Using the HVS equipment it is possible to apply either single or dual wheel loading at a wheel load magnitudes between 30 to 110 kN. The applied wheels can travel in the longitudinal direction at a speed of up to 12 km/hr (Wiman and Erlingsson, 2008). The wheels are also able to move in transverse directions (for example to simulate traffic wander). Environmental influences, such as pavement temperature and ground water table, are controlled through add-on facilities.

The bituminous bound layers of two HVS pavement test structures (named SE09-1 and SE09-2 respectively) are studied in this paper. The test structures were constructed from a thin asphalt surface layer, binder layer, bituminous base, unbound granular base and a fine sand subgrade over a rigid bottom (Wiman, 2010). A modified or enhanced asphalt mix was used for the binder layer of SE09-1 where as conventional asphalt mix was used for SE09-2 structure. Various types of sensors were installed within different layers of the structures to measure the response and performance characteristics such as permanent deformation in the asphalt layers and surface deflection. As the objective of the test was to understand the influence of the modified binder layer, the vertical strain gauges were installed only in the asphalt layer. Figure 1 presents the cross sections of the test structures and the instrumentation.

The sensors were installed along the centreline of the structures. Two or three sensors of each type were installed, thus the measurement used in this paper represent the average measurements form the sensors.

A total of 550,000 load cycles was applied during the permanent deformation measurement as a part of the HVS test. A single wheel load of 60 kN at a tyre inflation pressure of 900 kPa was used during the test. The test was started at a pavement temperature of 10°C and after a load cycles of 150,000 the temperature was increased to 20°C. After another 300,000 load cycles, the temperature was raised to 30°C and the test continued for another 100,000 load cycles.

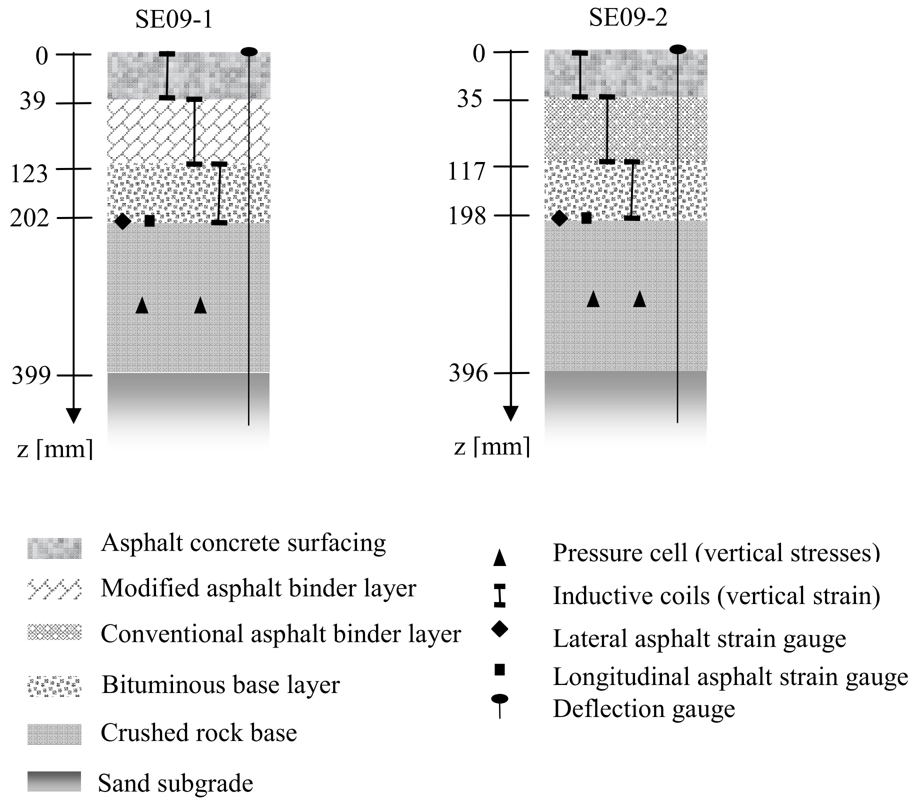

Figure 1: Cross sections of HVS test structures.

3.1 Material properties

Tables 1 and 2 present the selected material properties of the test structures. The stiffness of the asphalt layers at temperatures of 10°C is shown in the tables (Wiman, 2010). In the response analysis, all the asphalt layers and subgrade layer were assumed as linear elastic materials. On the other hand the unbound granular layers (crushed rock) were considered as nonlinear elastic materials with stress dependent stiffness according to Equation 3 (ARA, 2004; Uzan, 1985):

$$M_r = k_1 p_a \left(\frac{3p}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \tag{3}$$

where M_r denotes the resilient modulus, p denotes hydrostatic stress, which also includes the self-weight of the material and the lateral earth pressure and is given by $p = (\sigma_{kk} + (1+2k_o)\gamma z)/3$, where σ_{kk} is the normal stress tensor, k_o is the coefficient of lateral earth pressure, γ is the unit weight of the material and z is the depth, p_a = 100 kPa; k_1, k_2 and k_3 are material constants and τ_{oct} is the octahedral shear stress.

Table 1: Properties of SE09-1 test structure with modified binder layer.

Layer	Thickness	Stiffness/Resilient modulus				Poisson's ratio	Unit weight
	h [mm]	E [MPa]	k_1 [-]	k_2 [-]	k_3 [-]	v [-]	γ [kN/m^3]
Asphalt surface	39	7614	-	-	-	0.35	25.0
Modified Binder	84	7962	-	-	-	0.35	25.0
Bituminous base	79	5423	-	-	-	0.35	25.0
Crushed rock	197	-	1500	0.5	0	0.35	23.0
Sand subgrade	-	140	-	-	-	0.35	17.3

Table 2: Properties of SE09-2 test structure with conventional binder layer.

Layer	Thickness	Stiffness/Resilient modulus				Poisson's ratio	Unit weight
	h [mm]	E [MPa]	k_1 [-]	k_2 [-]	k_3 [-]	v [-]	γ [kN/m^3]
Asphalt surface	35	7614	-	-	-	0.35	25.0
Conventional Binder	82	7837	-	-	-	0.35	25.0
Bituminous base	81	5423	-	-	-	0.35	25.0
Crushed rock	198	-	1500	0.5	0	0.35	23.0
Sand subgrade	-	140	-	-	-	0.35	17.3

The stiffness's of the asphalt layers were obtained using indirect tensile testing (IDT). The tests were carried our for test temperatures of 5, 10 and 20°C (Wiman, 2010). The stiffness's of the asphalt layers at a temperature of 30°C was calculated from:

$$E(T) = E_{ref} e^{b\left(T - T_{ref}\right)} \tag{4}$$

where E_{ref} denotes the stiffens of the asphalt layer at a reference temperature T_{ref} = 10°C and b is a regression constant.

3.2 Asphalt binder properties

The HVS structures were constructed of a surface layer of ABT 70/100 penetration grading. As it was mentioned in the preceding sections, different types of asphalt binder layers were used for the test structures. The binder layer of the SE09-1 HVS test structure was made of polymer modified binder layer of ABb 70/100 penetration grading. While for the SE09-2 test structure, a conventional binder layer of ABb 70/100 was used. The road base layers of both structures were made of an open graded mix AG 160/220 penetration grading. A detailed description of the material properties can be found in (Wiman, 2010; Oscarsson, 2011). The binder and mix properties are shown in Table 3.

Table 3: Binder characteristics of the different asphalt layers.

Layer	Max Aggregate size (mm)	Binder content % by weight	Air void %
Asphalt surface course	11	6.5	4.34
Conventional Binder course	22	5	6.77
Modified Binder course	22	5	6.75
Bituminous base	22	4.5	7.88

4 TRAFFIC WANDERING

Pavement structures are subjected to traffic flow that does not follow a straight course or channelized traffic but rather they are subjected to wandering traffic, it is therefore necessary to take the effect of wandering pattern into account both during testing and modelling the behaviour of the HVS test structures. Therefore during the testing phase, the applied wheel load was moved laterally over a width of 70 cm at an interval of 5 cm either side of the centre of the wheel. For each loading position, a frequency (load repetition) as shown in Figure 2 (the bar chart plot) was used.

Figure 2 also presented the loading frequency derived from a normal distribution with standard deviation σ = 10.7 cm and mean μ = 0 cm (shown as broken line). As can be observed from the two graphs, the loading frequency used during the test was very close to those frequencies derived from normal probability distribution function.

To illustrate how the traffic wandering was incorporated, consider calculating permanent deformation at the lateral position $x = 0$ as shown in Figure 2. First the responses at $x = 0$ due to loads located at $x = -35$ cm, $x = -20$ cm…, up to $x = 35$ cm were computed. The computed responses and their corresponding frequency from the distribution curve in Figure 2 were then used to estimate the permanent strain contribution for the period under consideration. These contributions were then added together to find the total permanent strain at $x = 0$ due to all the loads. Similarly procedures were adopted to calculate the permanent strain at other lateral positions. A time hardening (Lytton et al., 1993) procedure was applied to combine the permanent deformation contributions from different stress levels.

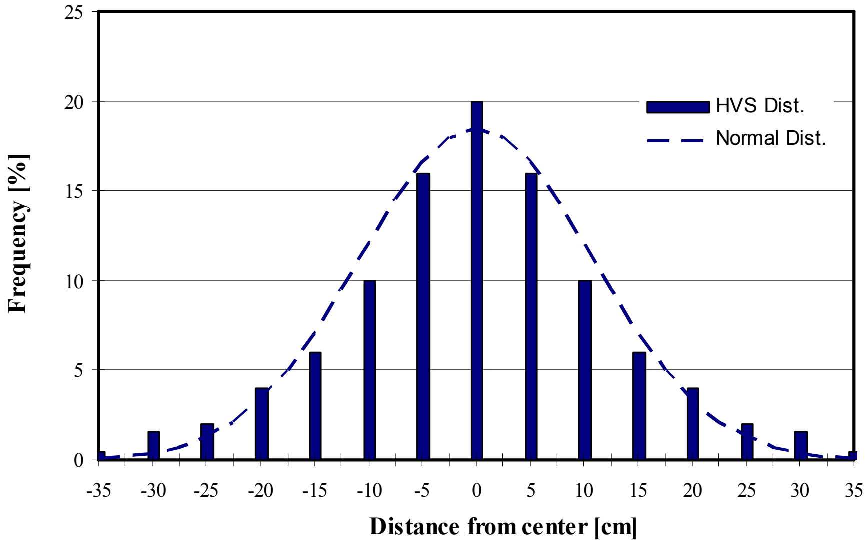

Figure 2: Lateral wander distribution of HVS test section and its approximation using normal probability distribution function.

5 RESULTS AND DISCUSSION

Figures 3 to 5 present the modelled and measured permanent deformations for the modified and conventional test structures. As can be observed from the figures, the M-E permanent deformation model captures the permanent deformation behaviour of the asphalt layers at 10 and 20°C pavement temperatures; however the model over-predicted the permanent deformation for pavement temperature of 30°C. The starting values of the model parameters a_1, a_2 and a_3 according to Equation 2 were obtained from previous literature studies (Erlingsson, 2010). However the a_1 parameter was optimized to obtain a good fit with the measured permanent deformations. a_2 and a_3 were kept constant. A least square technique was employed to estimate a_1 and the model parameters are shown in Table 4.

One of the reasons for the observed discrepancies in the measured and predicted permanent deformations may be due to the fact that asphalt mixtures exhibit unique characteristics of both viscous and elastic properties at higher temperature. However, for the purpose of response modelling in this study, all the asphalt layers were considered as linear elastic materials at all temperatures. The assumption of linear elastic behaviour for asphalt materials holds true at low pavement temperature and high loading frequency as indicated by several researches. The results in this paper also tend to confirm this assumption that at low temperature, linear elastic response modelling is sufficient to capture the permanent deformation behaviour using the MEPGD permanent deformation model.

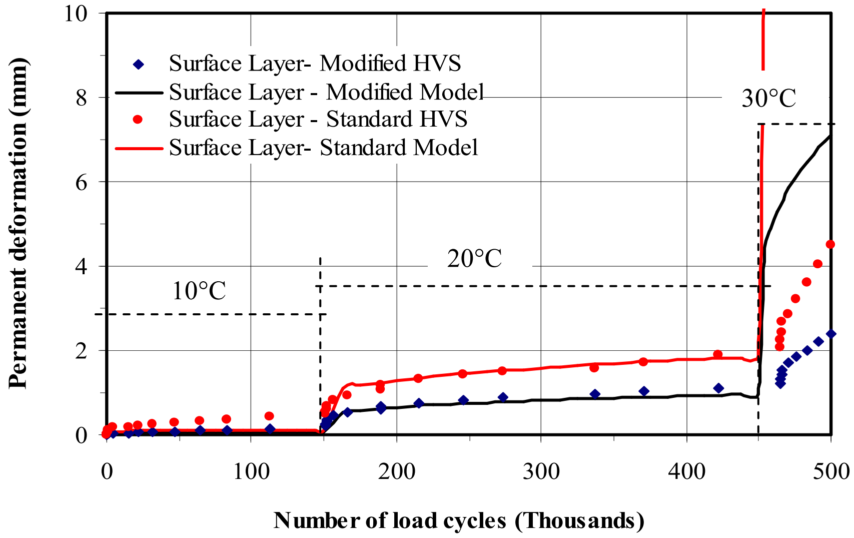

Figure 3: Measured and modelled permanent deformation for the surface layer.

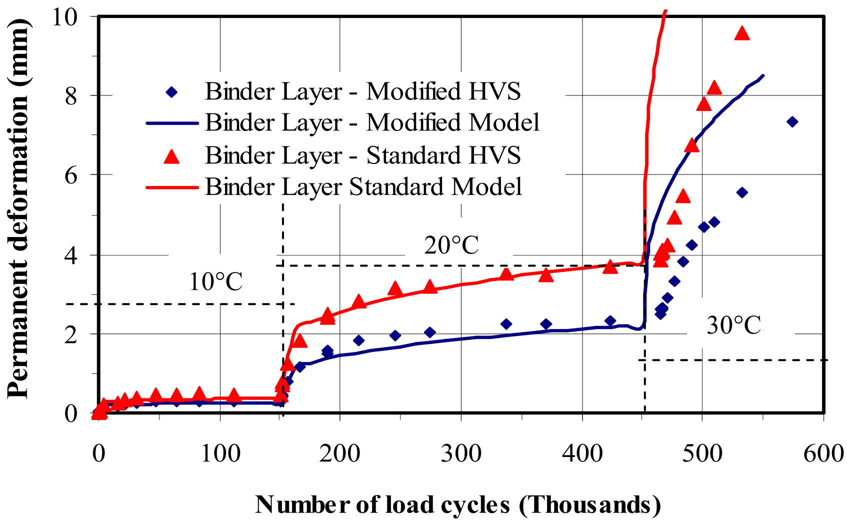

Figure 4: Measured and modelled permanent deformation for the binder layer.

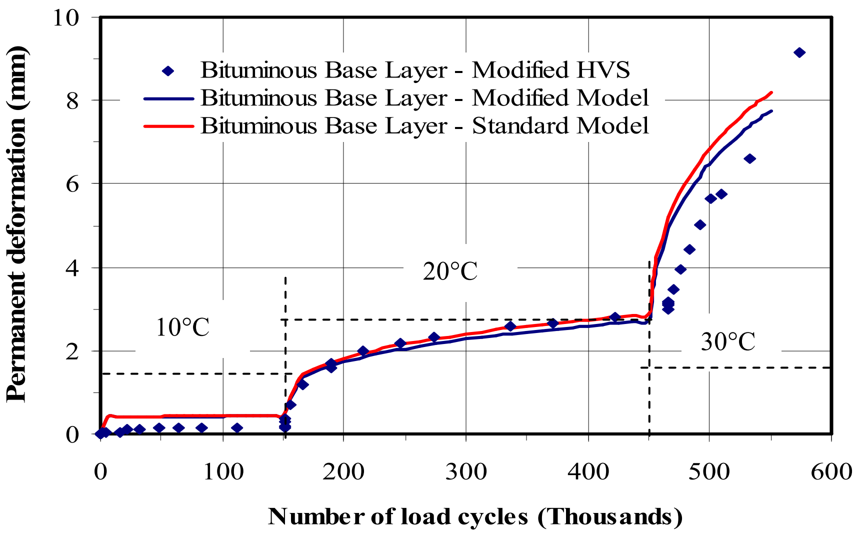

Figure 5: Measured and modelled permanent deformation for the bituminous base layer.

Furthermore, as described in the preceding sections, the stiffness of the different asphalt layers were measured using IDT at temperatures of 5, 10 and 20°C and Equation 4 was used to estimate the temperature at 30°C. Therefore this might also be the other reason for the discrepancies observed at higher temperature. Thus, a more rigorous characterization of the asphalt layers by using master-curve derived from frequency-sweep dynamic stiffness and/or shear modulus tests may be necessary to improve the accuracy at higher temperature and estimate model parameters.

The effect of aging of the binder was not considered in the modelling because the test was carried out in a rather short period of time so the influence of binder aging can be neglected.

The estimated model parameters for the M-E permanent deformation model, shown in Equation 2, for the different asphalt layers are shown in Table 4.

Table 4: Permanent deformation parameters for the MEPDG model.

Layer	a_1	a_2	a_3
Surface layer – Modified	0.200	1.85	0.27
Surface layer – Conventional	0.560	1.85	0.27
Binder layer – Modified	0.032	1.85	0.27
Binder layer – Conventional	0.048	1.85	0.27
Bituminous base layer Modified	0.024	1.85	0.27
Bituminous base layer Conventional	0.024	1.85	0.27

6 CONCLUSIONS

An M-E permanent deformation model for asphalt layer was evaluated in this paper. In the modelling, the effect of traffic wandering and the stress dependency of the unbound granular layers were considered. However, the asphalt and the fine sand subgrade layers were assumed as linear elastic materials.

The results showed that with appropriate calibration parameters, the model captures the permanent deformation characteristics of the asphalt layers at 10 and 20°C. However for higher temperature, 30°C, the approach over-predicted the permanent deformation.

The model parameters for the three types of asphalt mixtures were also estimated. The factors were calculated based on measurements from HVS test which might not fully represent the actual field situations where the climate and loading conditions vary continuously.

In this paper, all the asphalt layers were characterized as linear elastic materials which holds true for materials loaded at low temperature and high loading frequency. Thus a more realistic characterization of asphalt materials, such as viscoelastic or viscoplastic modelling, should be employed to capture the permanent deformation behaviour at higher temperature. Moreover, more rigorous testing methods such as frequency-sweep dynamic modulus tests should be used to estimate the stiffness of the asphalt mixtures at different temperature and loading frequencies. Future work of this study encompasses a more detailed characterization of the different asphalt layer and verifying the model parameters using other types of accelerated pavement testing performed on similar mixtures.

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