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Case history on a railway tunnel in soft rock (Morocco)

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ABSTRACT: Ras R'Mel Moroccan tunnel is a 2.6 km long and 60 m^2 section single track railway tunnel. It was excavated in a complex geological context of highly heterogeneous and highly deformable soft rock. The purpose of this paper is first to describe the methodology used for specific design of support and lining. The deformation monitoring process applied during construction will then be detailed, and an analysis of the deformations measured will be given, making a comparison with design calculations results. Main concerns were potential face instability and high tunnel deformations.

1 PROJECT DESCRIPTION

The Ras R'Mel tunnel presented in this case-history is a single track railway tunnel, part of the new railway line being constructed in Morocco between the town of Tangier and its new Mediterranean harbour, located on the coast about 30 km north-east of the city. The tunnel itself is located some 15 km east of Tangier, near the location of Ras R'Mel. Western portal is called "Tangier portal", eastern one is named "Ras R'Mel portal". Kilometric points on the railway line increase from Tangier portal (PK 26+841) to Ras R'Mel portal (PK 29+445).

Chinese company TEC Engineering won the contract for construction of the tunnel, which also included construction of a second tunnel, shorter (600 m long) but in a similar geotechnical context, and construction of the railway platform in between the two tunnels. For both tunnels, following a previous collaboration on the Meknes tunnel project (another Moroccan railway tunnel, the study of which was described in a previous paper for AFTES October 2005 international congress), TEC Engineering chose French engineering companies SETEC TPI and TERRASOL, subsidiaries of French group SETEC, as its consulting engineers for specific design of support and lining, and for deformation analysis during construction. This paper focuses on design and construction issues regarding Ras R'Mel tunnel.

The tunnel, of a 2600 m length and a 60 m^2 section, was excavated in highly heterogeneous flyschs, with up to 150 m depth in its central part (Cf. Fig. 2). The excavation was given a horse-shoe shape of a 8.5 m height and a 7.6 m to 7.8 m width, depending on the lining thickness in order to preserve the ultimate circulation clearance (Cf. Fig. 1). Four zones with provisional enlarged sections were excavated, in order to allow entering and exiting trucks to cross during construction. The enlarged sections are of a 10.2 m width and a 9.4 m height. At final stage, the over-excavation of those zones will be filled with lining concrete.

The entrance portals were at a depth of only 10 m. As those portals were almost vertical, and as the quality of the pelites, classified as soft rocks, is rather poor, it was decided to stabilise the portals building two 22 m long "false tunnels" (one at each portal) using a cut and cover technique.

Indicative tunnel profile is given on Figure 2.

From each portal, and on a total length of around 1600 m, the excavation is at a depth lower than 50 m: great surface settlements were expected in this zone. In the central part of the tunnel, on a length of 1000 m, excavation is at a depth of about 150 m: low surface settlements but high convergence and tunnel settlements

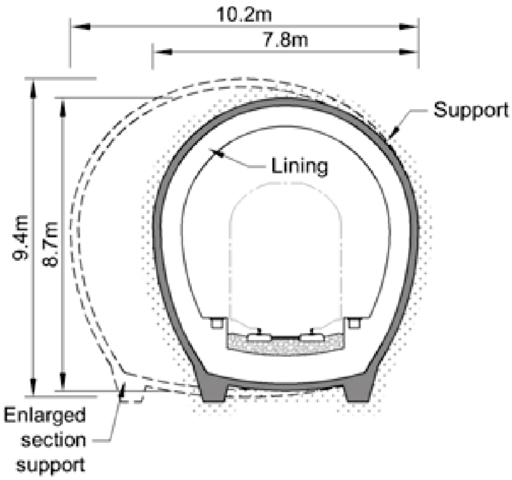

Figure 1. Type profiles (basic and enlarged cross-sections).

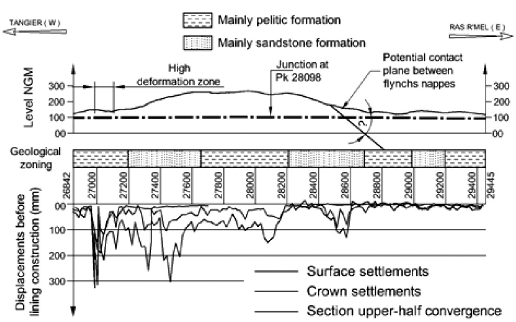

Figure 2. Tunnel profile and deformations measured before lining construction.

were expected in this zone. As the tunnel is in a countryside environment with no construction at surface, surface settlements were not expected to be of a major concern, but tunnel convergences were.

2 GEOTECHNICAL CONTEXT

Ras R'Mel tunnel is located in a geologically complex and highly tectonised zone. Two flyschs formations can be found, the older one (the Tisiren nappe, alternating levels of sandstone and pelite) being on top of the younger one (the Beni Ider Nappe, softer clayey-calcareous flysch) being a thrust-nappe. The contact plane between those formations slightly dips towards east. It was expected to be intersected by the tunnel but could not be firmly identified during construction. This contact is known to be intersected by several sub-vertical faults disturbing locally the lithology.

Table 1. Geomechanical characteristics.

Formation	Mainly pelite	Mainly sandstone	Fully sandstone
γ (kN/m^3)	24	24	24
UCS (MPa)	0.66	–	46
C_u (kPa)	200	500	–
φ_u (°)	0	0	–
C' (kPa)	20	100	1000
φ' (°)	28	30	35
E_0 (MPa)*	500	1000	1500
E_∞ (MPa)*	250	–	–
ν	0,3	0,3	0,3
σ_g (kPa)	200	0	0

* E_0 and E_∞ are respectively short and long term Young's moduli.

Furthermore, the geological formations are complex themselves: both flyschs formations consist of alternating levels of pelites (soft rock with Unconfined Compressive Strength (UCS < 1 MPa) and highly resistant sandstones (UCS = 30 to 40 MPa). Four (4) kinds of formations were then expected:

– non-altered, highly fractured pelites, with occasional breccia zones, Rock Quality Designation (RQD) ranging from 50% to 75%, and sandstones levels RQD, expressed as a percentage, is the summed length of core pieces greater than 10 cm measured for a 1 m long core pass; once excavated, the pelites get quickly altered;
– blocky sandstones, with occasional pelite levels;
– alternating levels of pelites and sandstones of low thickness, that can be found intricated in blocky sandstones;
– claystone breccia.

As blocky sandstones layers where not persistent, no continuity for those levels could be drawn between the borehole logs, and no accurate geological profile could be built for the project. It has then been decided, to propose an indicative geological zoning along the axis of the tunnel, for support and lining design of the project, pointing at three main categories of formations:

– formations mainly pelitic;
– formations mainly comprising sandstones;
– formations fully comprising sandstones;

Table 1 sums up the average mechanical characteristics given to these equivalent formations. Short term cohesion values for mainly pelitic formation are the updated values following analysis of first measures of tunnel deformations.

Potential swelling pressure σ_g in pelites is evaluated to 200 kPa, applying under lining invert.

Because of the thrust nappe context, high values for horizontal stresses were expected, which was taken into account in the design calculations setting parameter K_0 to 1 (horizontal stress equal to vertical one).

During construction, sandstone levels were confirmed non-persistent, which meant that the "Fully Sandstone" formation was non relevant. The two other formations describe soft rocks with rather weak characteristics: under expected geostatic stress state, the stability ratio σ_0/UCS ranges from 2 to 3,5 (>1) meaning that stress level is high enough for excavation to be unstable without support.

From a hydrogeological point of view, captive water of limited extent had been identified in the boreholes, sheltered by sandstone layers. The surrounding pelite matrix is little permeable. However, the highly fractured pelites and sandstones might allow limited water flows to reach the tunnel. It was then decided to protect tunnel lining with a watertight membrane on crown and shoulders.

3 PROJECT DESIGN

TERRASOL and SETEC TPI performed specific design studies for support and lining of the tunnel, as well as stability analysis of tunnel face. In order to control tunnel and surface deformations, a full face excavation method was considered.

Based on the indicative geotechnical zoning proposed for the tunnel, six calculation profiles were defined, named P1 to P6. Those profiles differed by the formation and depth taken into account for calculation (each profile is supposed to be composed of one single formation). Five additional profiles were studied, corresponding to potential enlarged section zones. The precise location of those zones would be defined precisely during construction, depending on real geological conditions.

3.1 *Tunnel face stability analysis*

Tunnel face stability analysis was performed using TERRASOL convergence-confinement code *TUNREN*. The code uses a C-Phi reduction method, based on the analytical model EXTRUSION developed by Wong et al. (1999). Hypothesis are the ones of convergence-confinement method (circular section, isotropic, homogeneous and infinite medium, uniform stress field), with two additional ones:

– tunnel face is supposed spherical, and stress-strain field follows a spherical symmetry;
– tunnel face heading is modelled by taking into account a decreasing radial pressure at tunnel face.

Material behaviour is supposed elasto-plastic, following Mohr-Coulomb or Tresca criterion.

From initial (c,φ) short-term values, mechanical characteristics of the ground are progressively reduced by a security factor F. For each set of mechanical characteristics, average radial strain is calculated at tunnel face, and results are presented via a chart giving tunnel face relative deformation ($\varepsilon_f = u_f/R$, where u_f is extrusion value and R excavation radius) versus security factor F. Stability is evaluated considering security factor values $F(\varepsilon_f = 2{,}5\%)$ and $F(\varepsilon_f = 5\%)$ giving relative deformations of respectively $\varepsilon_f = 2{,}5\%$ and $\varepsilon_f = 5\%$. Figure 3 shows an example of result chart, where $F(\varepsilon_f = 2{,}5\%) = 1.2$ and $F(\varepsilon_f = 5\%) \approx 1.4$.

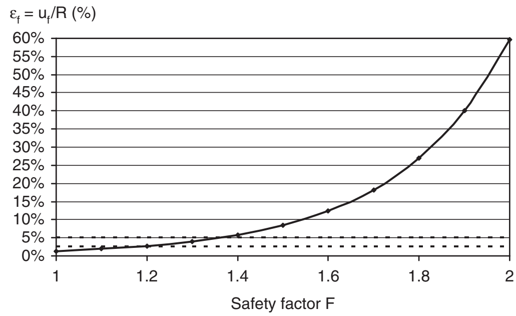

Figure 3. Tunnel face stability analysis for P1 profile (mainly pelitic formation at 50 m depth).

Calculation showed that tunnel face stability depends on tunnel depth. Whatever the formation considered, both security factors $F(\varepsilon_f = 2{,}5\%)$ and $F(\varepsilon_f = 5\%)$ were greater than 1 for depth lower than 50 m. For maximal depth (150 m), security factors where lower than 1. Sensitivity analysis showed that tunnel face became unstable under 60 to 70 m overburden. Ground behaviour appears to be limit-plastic at 50 m depth, and totally plastic at 150 m depth.

In order to perform construction using full-face excavation method, TERRASOL-SETEC TPI advised face reinforcement would then be necessary.

3.2 *Support and lining design*

The construction method chosen was a half-sections method. Support and lining, as defined in the contract, were as follows:

– *support*: all over the section, including invert, 5 cm shotcrete confinement layer, plus HEB 180 steel ribs settled in a 18 cm thick shotcrete layer; steel ribs spacing had to be defined by design calculations, and was expected to range from 0.75 m to 1.5 m depending on geological conditions;
– *lining*: watertight layer on crown of the tunnel, and 40 to 60 cm thick concrete reinforced at lateral walls and invert junction; reinforcement sections had to be defined by design calculations.

In order to estimate tunnel deformations and solicitations of lining, a staged calculation was run with finite elements model (FEM). Calculation

Table 2. FEM support calculations (after calculation profile name, P and S are respectively for "Mainly Pelite" and "Mainly sandstones", figure indicates depth).

Calculation profile	Surface settlement (mm)	Crown ground settlement (mm)	Horizontal convergences (mm)	
			Ground	Support
P1 (P, 50)	3.2	13	44	9
P2 (S, 50)	1.1	4.5	11	4
P3 (S, 150)	–	25.8	60	16
P4 (P, 150)	–	97.6	190	12
P5 (P, 10)	0.4	1.8	6.5	3
P6 (P, 30)	0.7	3	8	3

procedure was lead through TERRASOL convergence-confinement code TUNREN, and through French LCPC finite elements code CESAR. For each profile, calculation steps were as follows:

– Convergence-confinement calculations: estimation of the deconfined rate λ_0 after tunnel full-face excavation, using TUNREN code;
– FEM calculations: full-face excavation (deconfinement up to λ_0 value previously estimated, load applied to excavation walls set to $(\sigma_N = (1 - \lambda_0)\sigma_0)$; completion of support and final deconfinement $(\sigma_N = 0)$; completion of lining (mechanical characteristics are set to long-term values, swelling pressure is applied under invert and ground creeping is taken into account by option "EFD" of the calculation code).

Results were expressed as deformations and stress fields for support, and only as stress fields for lining. Table 2 sums up displacements results after support calculations.

Results showed that surface settlements were expected to be lower than 5 mm.

For a given formation, tunnel deformations were expected to grow with tunnel depths. For formations with mainly sandstones, maximal convergences in support and crown settlements are respectively 16 mm and 25.8 mm, which is rather limited. For mainly pelitic formations, they are of respectively 12 mm and 97.6 mm, which shows that at maximal depth, tunnel behaviour is completely plastic and deformations can become important. Numerical model indicates crown ground settlements about half as high as ground horizontal convergences for depth greater than 10 m.

Support calculations indicated that great deformations (of about 100 mm) could occur in support at depth greater than 50 m. Support completion would then be of a major concern regarding control of those deformations.

Lining calculations showed that only minimum reinforcement would be necessary at junction between lateral walls and invert.

Figure 4. Upper half-section excavation with central part of the face left in place (Tangier face, around PK 27+960).

4 CONSTRUCTION

Works began in February 2006, with access cut excavation. Tunnel excavation began in March 2006 at both portals. Junction of the two faces occurred at PK 28+098 on July the 5th of 2007, after 15 months. Average heading was of about 80 m per month at Tangier face, and of about 90 m per month at Ras R'Mel face. Ras R'Mel face appeared to be in better geological conditions than the other face, which allowed higher heading rate and lighter support: steel ribs could be spaced from 1,2 to 1,5 m in average, against 0,75 m in average at Tangier face.

TEC Engineering decided not to apply face reinforcement treatment. As tunnel-face was feared to be unstable, it was then decided to apply the same half-sections excavation method as used for Meknes tunnel. In order to control tunnel face stability, during excavation of section upper-half, and in zones with poor quality ground, central part of the face was left in place until upper-half support completion (Cf. Fig. 4).

Furthermore, as lower-half excavation and support completion occurred in average 4 days after upper-half excavation, behaviour of tunnel face was almost equivalent to a full face excavation, which helped control deformations.

Opening ranged from 0,75 m in zones with mainly pelites, to 1,5 m in zones with mainly sandstones, and was adapted to the deformations measured. A provisional formwork of wooden boards hold by steel bars prevented shotcrete loss during shotcrete projection, as can be seen on Figure 3.

A total of 4 crossing zones with enlarged section was excavated, 2 at each face.

As expected, main concern during construction was tunnel deformations due to excavation of soft deformable rocks under high stress level.

4.1 *Surface deformations monitoring and analysis*

From each portal, and on a total length of 1500 m, excavation is at a depth lower than 50 m, Ras R'Mel face even having to cross a zone at a depth of only 15 m. Surface settlements where then monitored. Every 20 m along tunnel axis, a monitoring cross-section was set using 5 surveyor's rods set perpendicularly to the tunnel axis (numbered from 1 to five, with 10 m to 15 m spacing, central rod number 3 being above tunnel axis). For a given monitoring section (named following the corresponding kilometric point of the tunnel), measurements started 2 to 4 weeks before tunnel face reached the given kilometric point. The frequency was one measure per day up to stabilisation, and one measure a week once stabilisation reached. Settlements in the centre part of the tunnel, at greatest depth (150 m), were not monitored.A chart showing final surface settlements measured before lining construction is presented on Figure 2.

4.1.1 *Ras R'Mel face*

At Ras R'Mel face, settlements for a given monitored section began when tunnel face was approximately 20 m ahead of the corresponding PK. Stabilisation was reached as tunnel face was 80 m further (approximately after 1 month). At tunnel axis, final settlements are of about 20 mm for the first 100 m after portal. Further on, they are inferior to 5 mm. No relation could be established between tunnel depth and surface settlements. The low settlements did not allow observation of relevant transverse settlement throughs, and the crossing of the very low depth zone, between PK 28+924 and PK 28+870, did not induce any increase of surface settlements.

Surface settlements measured were as expected after FEM calculation.

4.1.2 *Tangier face*

At Tangier face, settlements for a given monitored section began as tunnel face was about 20 to 40 m ahead of the corresponding PK. Stabilisation was reached when tunnel face was 80 to 120 m further (1 to 1.5 month later). Above tunnel axis, final settlements are rather high in the first 100 m after the portal (from 20 to 50 mm). These values, higher than the ones calculated, could be explained by the proximity of the access cut to the portal, and by the low depth of the tunnel in this zone. Nevertheless, as a 45 m overburden was reached, and settlements seemed to decrease, tunnel face hit a rather weak zone (from PK 26+978 to PK 27+140), in which surface settlements were much higher than expected (greater than 20 mm at tunnel axis, reaching up to 190 mm).

Figure 5 shows settlements evolution at PK 27+000. Tunnel face upper-half reached the corresponding section, when already 20 mm settlements had occurred at surface. The effect on surface

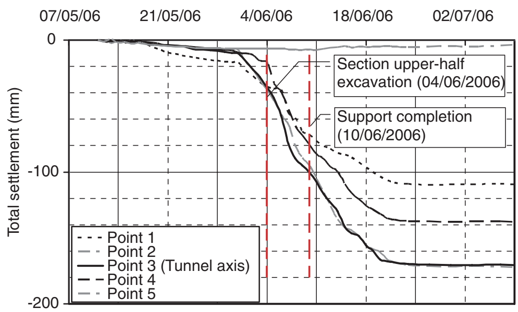

Figure 5. Settlements at PK 27+000 (Weak zone at Tangier face).

settlements of tunnel face lower-half reaching the section, can hardly be seen as no stabilisation had occurred at that moment. Stabilisation was reached, two weeks after completion of tunnel support. The final settlement value above tunnel axis is of 175 mm. Those settlements can be related to high tunnel deformations as will be discussed further on.

Final settlements for points 5 and 4, respectively compared with those for points 1 and 2, are lower. Almost no settlement occurs at point 5. This dissymmetry in transverse settlement through is relevant of what could be observed at Tangier face: surface terrain is naturally dipping south, so that the cross-section monitored is in direction of the dip. Points 1 and 2 are on the upper side of the dip, whereas points 4 and 5 are on the lower side. Crossing a weak zone exaggerated this phenomenon.

As no building was constructed above the tunnel, such high settlements were not of a major concern.

After this zone of high deformations, settlements are lower than 20 mm. From pk 27+140 to pk 27+340, surface settlements at tunnel axis increase from 7 mm to 17 mm, following depth increase from 40 m to 90 m. For greater PK, settlements decrease while depth increase. As the centre part of the tunnel is reached (with depth of 150 m), final settlements are of a 5 mm average, which fits the settlements calculated by the FEM simulations, and is compatible with measures relative to Ras R'Mel face.

Surface settlements were then partly dependent on tunnel depth, and partly on lithology (greater settlements in highly deformable soft rock).

4.2 *Tunnel deformations monitoring and analysis*

As said before, tunnel deformations were expected to be of a major concern. Every 20 m along tunnel axis, a monitored cross section was established after support completion, equipped with 5 measuring targets. It allowed measurements in the support of crown and side wall settlements, and of section upper and lower-half convergence.

Due to construction constraints, convergence measurements are of questionable reliability. However, it clearly appears that final measured convergences are highly variable from one section to another, depending on lithology rather than on tunnel depth. The convergence values reached range from 20 mm to more than 300 mm, which is far greater than estimated by calculation. Furthermore, differences appear between the two tunnel faces, Ras R'Mel face once again showing lower deformations than Tangier's. Figure 2 shows tunnel deformations.

At both faces, tunnel deformations appeared to stop as soon as support was completed, closing the excavated sections. Later creeping could be identified as lining was being constructed.

4.2.1 *Ras R'Mel face*

From Ras R'Mel portal, up to junction PK, tunnel deformations were limited, section upper-half deformations ranging from 10 mm divergence to 30 mm convergence, crown settlements being of a comparable size. Those values are limited, but higher than calculated for corresponding profiles (P1, P2, P3, P5 and P6).

From PK 28+620 up to PK 28+400, tunnel deformations are higher, reaching 130 mm convergences and 78 mm crown settlements. In this zone, crown settlement is about half of convergences. This local increase in deformations should be related to decreasing quality of excavated rock rather than to an increase of overburden. The contact plane between the two flyschs formations might have been crossed along with this change of rock quality, but could not be identified firmly.

Approaching PK of faces junction, the deformations increased and reached values consistent with the ones measured at the other face.

4.2.2 *Tangier face*

As shown for surface settlements, after a zone with rather high deformations due to perturbations induced by the vicinity of Tangier portal, tunnel suddenly hit a zone of very high deformations at limited depth (around 50 m), located between PK 26+978 and PK 27+140.

High convergences in the section upper-half occurred, greater than 100 mm and reaching 330 mm. Crown settlements were of values half as high. Trying to secure the zone, it appeared that deformations stopped as soon as section's support was completed. Final stabilisation was reached within one month after section opening, although later creeping could be identified locally causing support deformations. The values measured overcome by far the values calculated for support deformations. They are thought to be linked with local mechanical characteristics for the rock lower than expected.

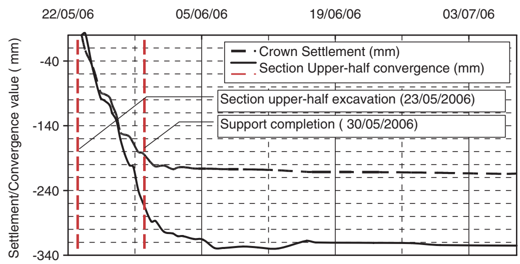

Figure 6. Greatest tunnel deformations, measured at PK 26+978.

It was feared that deformations would become unacceptable when reaching maximum depth and excavation would become highly unstable. As soon as it became obvious that support section completion blocked further deformations, it has been decided to go on construction with the same method, operating excavation carefully, and keeping opening and steel-ribs spacing at only 0.75 m as long as necessary.

Figure 6 shows the evolution of tunnel greatest deformations, which occurred at PK 26+978 (Stabilisation was reached by mid-June).

The high deformations zone goes up to PK 27+780, where greatest depth is reached. From this PK, and up to the junction of tunnel faces, deformations are still important (about 90 to 100 mm in section upper-half convergences, and 50 mm in crown settlements) but lower than the previous zone. This is supposed to be linked with depth increase, and a better stiffness of soft rock mass. The deformations measured in this zone, are higher than convergences calculated for the corresponding profile (P4).

Deformations values increased as junction section approached, reaching values compatible with the ones observed at Ras R'Mel face.

From a general point of view, it is thought that part of the deformations calculated as "ground deformations before support completion" (theoretically non-measurable), are measured as upper-half support convergences, because of the half-sections excavation method used, (measures begin as section excavation is not completed, and upper-half support is not totally blocked as no invert is constructed at that stage). This could explain why measured tunnel deformations are greater that the ones calculated. Section behaviour is then intermediate between full-face (as calculated in specific design studies) and half-section excavation.

Face potential instability had been pointed at by calculation. The cautious excavation method chosen allowed no collapse to occur on Ras R'mel tunnel works. The other tunnel for the railway project (Sidi Ali tunnel), operated by the same team of company in similar context but with slightly better geological conditions did not receive such cautious treatment at tunnel face. Tunnel face collapsed on 31/07/2006, which shows that TEC Engineering method was appropriate regarding Ras R'Mel tunnel face.

4.2.3 *Zones with enlarged section*

The four zones with enlarged section did not induce greater deformation compared to surrounding basic sections.

5 CONCLUSION

Ras R'Mel tunnel had to be excavated in highly variable geological formations, highly deformable soft rocks, and at a high stress level. To control tunnel deformations and tunnel face potential instability, it was then necessary to use half sections excavation and, when necessary, divided section excavation for upper-half. This method derived from a method previously used at Meknes tunnel (Marocco), operated by the same team of companies. During construction, sections behaviour was intermediate between full-section and half-sections excavations.

Design studies gave a rough size range for surface and tunnel deformations, but several zones with tunnel high deformations were crossed, which required cautious excavation. Nevertheless, the excavation method used allowed to control deformations, that were blocked as soon as support was completed.

At time this paper was written, tunnel completion was programmed for mid-September 2007.

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